THE UNIVERSITY OF ALBERTA

THE DETERMINATION OF SCOUR BETWEEN BRIDGE EMBANKMENTS ON GRAVEL BED RIVERS

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

SPRING, 1972

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled THE DETERMINATION OF SCOUR BETWEEN BRIDGE EMBANKMENTS ON GRAVEL BED RIVERS submitted by Donald Bruce Tutt in partial fulfillment of the requirements for the degree of Master of Science.

ABSTRACT

Observation and analysis of scour in the vicinity of spill-through bridge embankments is presented. The data presented are the results of experiments performed on a large scale mobile-bed hydraulic model of a gravel river.

The analysis is based on numerics obtained from dimensional analysis of the basic parameters involved.

New criteria for the determination of scour depths in an open-channel gravel-bed constriction appear to be emerging. These are based on recent advances in the description of the sediment transport phenomena and the continuity equation.

These trends have led to recommendations for further study and analysis.

ACKNOWLEDGEMENTS

This thesis is based on research performed during the summer of 1969 and sponsored jointly by the National Research Council of Canada and the Alberta Research Council. Grateful acknowledgement is made for the generous financial assistance given by both of these organizations.

The writer thanks Professor A.W. Peterson for suggesting the topic and providing guidance throughout the work, to Dr. T. Blench for his editorial advice during the absence of Professor Peterson, to Mr. C.R. Neill of the Alberta Research Council for his advice and criticism, to Mr. J. Belicek for his assistance in the laboratory, and to Mrs. L.A. Stalker for typing the final manuscript.

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THE DETERMINATION OF SCOUR BETWEEN BRIDGE EMBANKMENTS ON GRAVEL BED RIVERS

CHAPTER I

INTRODUCTION AND LITERATURE REVIEW

The imposition of obstructions, either natural or man-made, on the flow patterns of a river will cause the river to re-adjust to a new equilibrium condition to minimize their effect. The process of achieving equilibrium can involve either scour or deposition of the bed material. Scour, for the purpose of this report, is defined as the erosion of bed material caused by the enhancement of the channel discharge intensity due to a constricting of the normal channel width at any or all discharges. Bridge embankments and piers are common types of obstruction and many bridge failures have been reported due to the undermining of foundation piers and abutments.

At present, there is no generally accepted method of estimating the maximum depth of scour to be expected given the flow conditions and the geometry of a gravel river. This work defines some of the important parameters that pertain to the development and extent of scour at bridge constrictions on gravel rivers and uses laboratory data to determine if any practical methods exist for determining the amount of river bed degradation that would occur in an artificially constructed section during high discharges.

Neill (1964) has suggested the terminology depth of scour and

total scoured depth in reference to problems of river bed scour. He defined "depth of scour" as the depth of material removed during the scouring process measured below a datum level which was defined as "the normal low-water bed", and "total scoured depth" as "the depth from the water surface to the scoured bed". The present author accepts these definitions but also uses the term "equilibrium depth" which is equivalent to "total scoured depth"

Review of Previous Investigations

Previous investigations on scour in bridge waterways have centered on three basic approaches to the problem.

1. Field Investigations
2. Laboratory Investigations
3. Analytical Techniques

Some of the important work in each of these areas will be reviewed as a prerequisite to further study. A summary of existing equilibrium depth formulae is given in TABLE 1.

1. Field Investigations

Moulton, Belcher and Butler, (1957), have documented several bridge failures after a severe flood. A formula was obtained relating the average depth of scour to the difference between the sediment load in the approach channel and the transport capacity under the bridge. As most of their observations were made after the flood, they probably do not give maximum values for scour. The formula is not readily transferrable to other situations as the bed material was vaguely described.

Herbich and Brennan, (1967), used a regression analysis on field scour data from a number of bridge crossings in Southern Ontario after moderate floods. For some of the data, the slopes obtained differ substantially with the experience of the present author and that of Dickinson, (1968).

Kellerhals, (1967), derived regime type formulae for the stable dimensions of gravel channels. Most of the channels analyzed were immediately downstream of natural lakes and as such were known to have low bed load transport rates. In addition to formulae for the natural channel dimensions, he also presented formulae for the case when the channel width is fixed. TABLE 1 gives only this latter formula for depth determination.

The Regime Theory, based on engineering experience gained on the Indian Sub-Continent, has resulted in formulae for the depth of scour based on observation of large sand bed rivers. Inglis, (1949), compared the depth of local scour with the regime depth in the contracted section based on Lacey's work. The conclusion drawn was that the maximum depth of scour could be expressed as a multiple of canalized depth at flood discharge, with the multiple depending on the geometry of the situation. Blench, (1969), has also followed this approach.

Neill, (1964), has published a report on the nature of scour. This report describes the scour process and comments on the present methods of scour depth determination.

TABLE 1

SUMMARY OF EXISTING EQUILIBRIUM DEPTH FORMULAE

AUTHOR AND FORMULA	COMMENTS
Neill, derived from Lacey (1964) $D_M = 0.9 \dfrac{q_L^{2/3}}{f^{1/3}}$	Apply a factor dependent on river geometry for maximum depths D_M - mean depth (in terms of surface width) A/W_s q_L - c.f.s./ft. surface width f - silt factor - defined for duned sand only W_s - surface width
Kellerhals (1967) $d_* = .266\ Q_d^{0.8} K_s^{-.12}\ W_s^{-.8}$	d_* - depth in terms of surface width A/W_s Q_d - dominant discharge K_s - equivalent sand grain roughness W_s - water surface width of channel
Blench (1969) $d = \dfrac{q_B^{2/3}}{F_b^{1/3}}$	Applicable in general d - flow area/width at half depth q_B - c.f.s./ft. at half depth F_b - bed factor, varies with phase and size of bed material
Laursen (1962) $\dfrac{y_2}{y_1} = \left[\dfrac{B_1}{B_2}\right]^P$ $\dfrac{y_2}{y_1} = \left[\dfrac{Q_t}{Q_c}\right]^{6/7}$	Based on models y - flow depth; 1 - constricted B - channel width; 2 - unconstricted Q - discharge; t - total channel P - factor 0.5-0.69; c - main channel
Moulton et al (1957) $V_s = \dfrac{7}{C}\ \dfrac{Q}{R_s}$	Derived from prototype V_s - volume of scour hole Q - discharge R_s - relative transport capacity C - Chezy coefficient

Recent advances in the field of automatic scoured depth recorders should make more field data available for analysis in the future, (Basson, 1963; Corry and Sager, 1968; Sanden and Neill, 1963).

2. Laboratory Investigations

Laboratory data on scour investigations on sand bed channels are plentiful in the literature but data pertaining to models of gravel rivers are rare. Because of phase differences, sand bed river models yield answers that are usually only qualitative, while models of coarse gravel bed rivers may be expected to yield reasonably good quantitative as well as qualitative answers.

Lane, E.W., (1919), was one of the first to conduct a laboratory study of scouring action in constrictions. He was interested in contractions as a method of discharge measurement and gave a series of formulae for vertical contractions in terms of head loss. The bed material used was described as "gravel and silt".

Laursen and Toch, (1953), qualitatively studied many aspects of the scour problem around bridge piers and constrictions. This work was one of the first to study scour both with and without upstream sediment supply in the approach flow. The median sizes of the uniformly graded sediment were 0.46, 0.58, 1.60, and 2.20 millimeters. They concluded that:

- The depth of scour is a function of the flow depth.
- The depth of scour is a function of the contraction of the flow section.
- The transport capacity is a function of the flow characteristics.

In any reach of river, the transport capacity must be equal to the material being transported, otherwise scour or deposition must occur until a balance is achieved.

Komura, (1966), experimentally studied the depth of scour in long constrictuions. The results were used to describe the scour at real bridge abutments, through the use of experimentally determined coefficients. The bed materials used had median sizes of 0.70, 0.55 and 0.35 millimeters by weight. The conclusion of interest here was that when the upstream bed charge was zero, the equilibrium depth of scour depended on the relative sediment size; when the upstream sediment charge was positive then the equilibrium scour depth was only slightly dependent on the relative sediment size.

Liu, Chang and Skinner, (1961), did a comprehensive model study of scour for different bed materials in the sand range, (median diameters by weight, 0.65 and 0.56 millimeters) and different construction geometries. Experimental relationships were obtained pertaining to the depths of scour under various flow conditions and geometries.

3. Analytical Techniques

The development of comprehensive mathematical models for the prediction of scour has remained at a very primitive stage. This is due to the extreme difficulties of analytically describing the component parts of the complex problem, namely:

- the water sediment complex,
- the boundary conditions,

- The development of 3-dimensional boundary layers and turbulence due to the merging flow near the abutments.

The authors of the following publications have given their attention to some of the individual theoretical aspects outlined above.

Moore and Masch, (1963), have advanced a scour theory from potential flow considerations. It is known that the deepest local scour holes tend to form on the upstream side of vertical piers and guide banks. The authors have explained this in terms of potential flow theory by integration of the Euler equation for the case of a parabolic velocity distribution. This integration yields the result that there is a vertical velocity component along the stagnation line of the pier that is theoretically equal to the surface velocity. Although potential flow theory applies only to inviscid fluids, which water is not, the theory has been used as an aid through laboratory experiments for explaining and making proposals for the control of local scour around bridge piers.

Vinje, (1967), and Thompson, (1966), have studied the effects of vortices on the movement of bed material. Their results show that:

- Saltation of stones is an effect of impulsive forces;
- Strong impulsive forces in streams are an effect of eddies in turbulent water;
- The effective roughness of the bed determines the scale of the eddies.

Bradley, (1960), has studied the effects of backwater caused

by bridge waterways. The backwater is caused by the constriction of the flow area. The presence of a constriction causes larger energy losses to the flow than would exist if the constriction were not there. A larger head is therefore required to pass the water through the constriction and this extra energy is obtained by the "damming" up of the water behind the constriction. It is this extra head that is used in reshaping the channel bottom in the vicinity of constrictions.

CHAPTER II

METHOD OF APPROACH TO THE PROBLEM AND DOMENSIONAL ANALYSIS

A river reach in equilibrium at all points implies that there is continuity of flow within the reach and continuity of bed load charge within the reach. If an obstruction, for an example, a bridge embankment, is placed within this reach, the river will no longer be in equilibrium and will tend towards a new equilibrium condition. Initially, there will be some storage of water due to the backwater effect caused by the embankments, and there will be a larger sediment charge moving downstream from the constricted reach than moving towards it from upstream due to the initially enhanced capacity of the water flowing through the contracted reach to carry a sediment load. In time, however, there will be an equilibrium of sediment charge transported at all points within the affected reach. Then, if it is possible to describe the hydraulic and sediment transport parameters of both the constricted and unconstricted channel, a relationship for determining scoured depths should evolve which can be evaluated by means of laboratory and field testing.

Dimensional Analysis

The dimensional analysis is given in terms of the unit discharge q which is the discharge per foot width of the channel. The assumptions used in this analysis are that at equilibrium there will be a continuity

of both flow and sediment charge and that quasi uniform flow conditions prevail throughout the entire channel. It is recognized that there are limitations to this last assumption in the contracted reach. The flow depth, h, is a function of the following variables,

$$h = \phi_1 \left[q, C, \rho_s, \rho_f, D_n, \alpha, \beta, \nu, g, b \right] \tag{1}$$

where b = breadth of channel

h = flow depth

q = discharge per foot width

C = sediment charge

ρ_s = density of sediment

ρ_f = density of fluid

D_n = bed material bulk sample particle size of which n% are finer by weight

α = shape factor of sediment gradation curve

β = shape factor of sediment particles

ν = kinematic viscosity of fluid

g = acceleration due to gravity

These terms can be arranged into the following numerics:

$$0 = \phi_2 \left[\frac{h}{D_n}, \frac{q^2}{gD_n^3}, \frac{\rho_s}{\rho_f}, \alpha, \beta, C, \frac{q}{\nu}, \frac{b}{h} \right] \tag{2}$$

Equation (2) may be simplified if:-

- the shape factor α of the sediment gradation curve is practically the same for all sediments considered;
- the shape factor β of the sediment particles is the same;
- the Reynolds Number, $\frac{q}{\nu}$ has negligible effect if, as argued by Yalin, (1965) the flow is fully rough turbulent;

- the breadth to depth ratio $\frac{b}{h}$ has been shown by Blench, (1969), to occur as a very small power and so for normal shaped channels ($\frac{b}{h} > 5$) can be ignored:
- the relative specific gravity of the bed material $\frac{\rho_s}{\rho_f}$ is the same for all water-sediment mixtures.

Equation (2) can then be reduced to the following:

$$0 = \phi_3 \left[\frac{h}{D_n}, \frac{q^2}{gD_n^3}, C \right] \qquad (3)$$

CHAPTER III

EXPERIMENTAL APPARATUS AND PROCEDURE

Apparatus

A model of a gravel bed river was constructed with plan and typical cross-section as shown in FIGURES 1 and 2. It is thought that this model would simulate a typical river of about 160-foot breadth. This would indicate a scale ratio, model to prototype, of about 1 to 40.

A natural gravel having an approximate log normal distribution and a median diameter of 1 millimeter was used as bed material for both the main channel and flood plain portions of the model. The original bed material gradation curve is shown in FIGURE A-2.

PLATES 1 and 2 are general views of the model showing rails for measurement, side slope protection, and the general model layout. The method employed for longitudinal control was the use of survey stations. Station 0 was at the upstream end of the channel and each station corresponds to a distance of 1 foot along the channel center line. The top of the rails was set at an arbitrary elevation of 10.500 feet. At high overbank flows, there was some disturbance of the flow patterns by the rail support posts but this effect was thought to be unimportant.

Water discharge was measured by means of a rectangular weir and

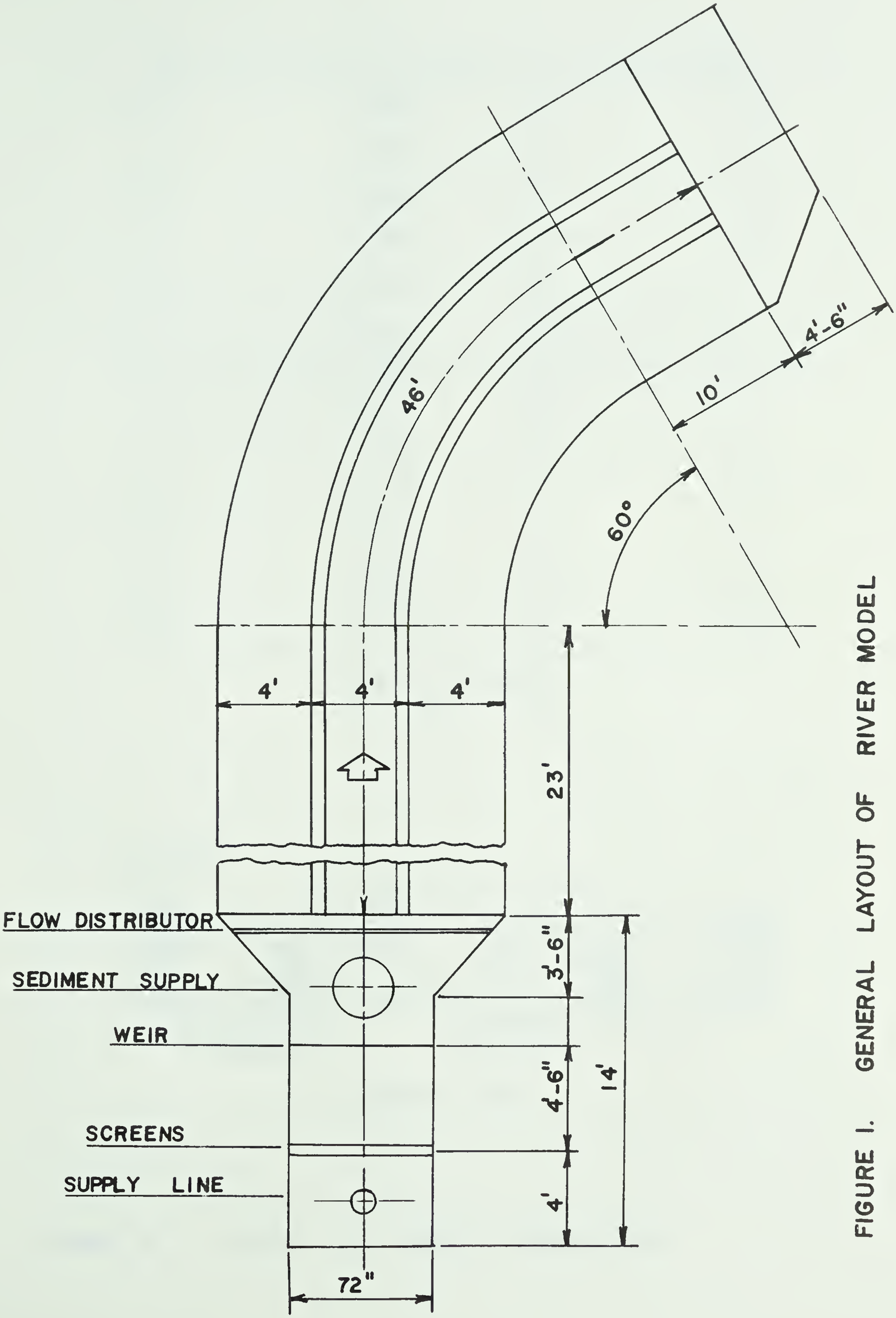


FIGURE 1. GENERAL LAYOUT OF RIVER MODEL

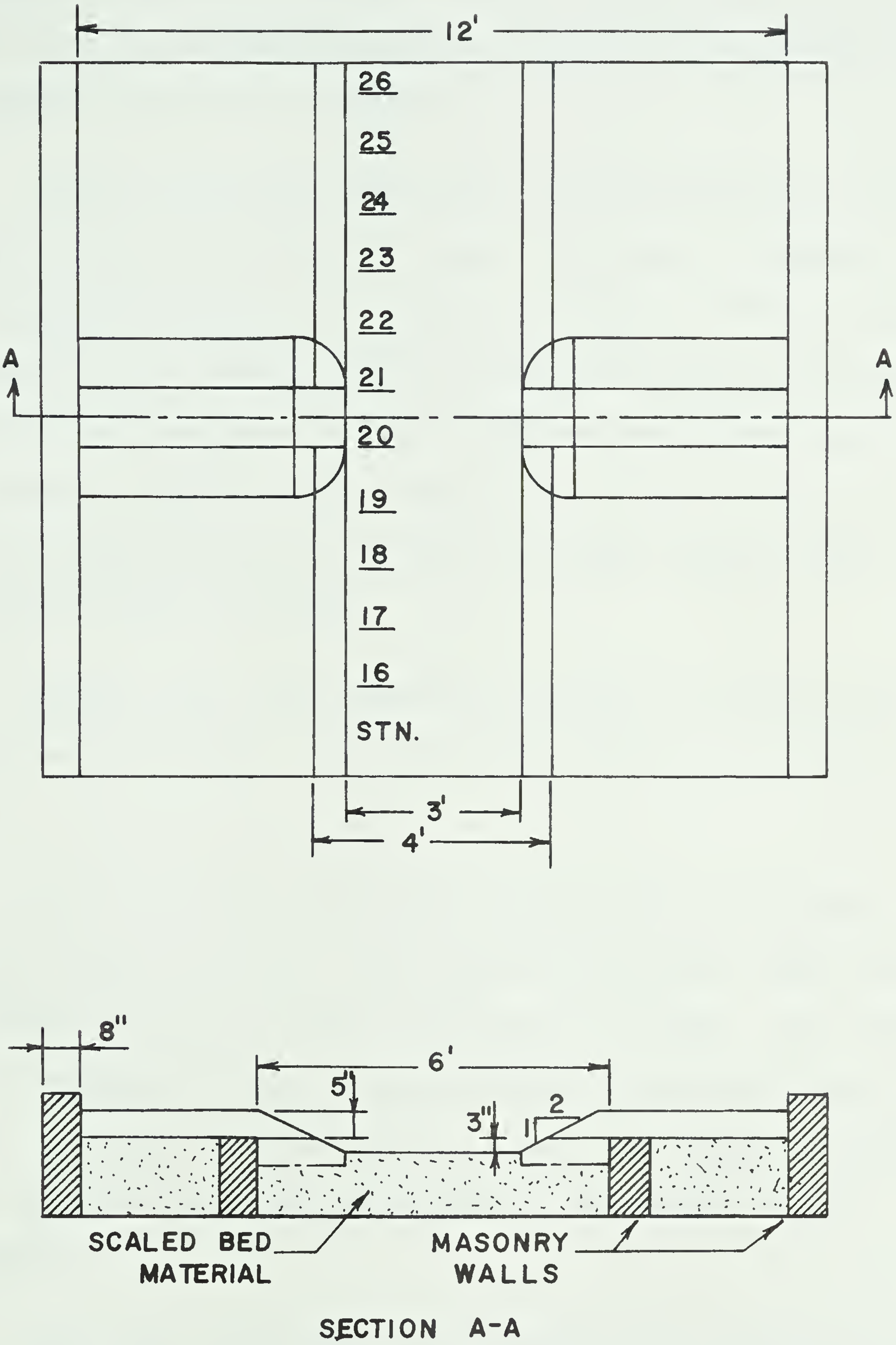


FIGURE 2. LAYOUT OF MODEL EMBANKMENT

point gauge and calibrated using the method of Kindsvater and Carter, (1957). This was checked with numerous velocity area measurements and the agreement was within a few percent.

Initial calculations showed that it would be necessary to add sediment to the flow, especially at higher discharges, to prevent a general degradation of the bed. Sediment was added to the flow at the upstream end of the flume by means of a vibrating hopper which was calibrated for different rates of feed at different weir discharges. The large amount of sediment used in each run necessitated the use of a gas-fired sediment drying unit as only dry sediment could be added with the vibrating feeder and hopper.

Expanded metal bank protection was found to be necessary to keep the designed granular side slopes from sloughing into the main channel.

Bed cross-sections were taken and plotted by an electrical resistance type bed plotter. Water surface profiles were taken with a point gauge which could be read to the nearest .001 foot. Point velocities were measured using a miniature current meter and a dekatron counter. All point velocities were 10 second time averages. Vegetable dye was used to observe the flow patterns in the vicinity of the embankments.

Due to time limitations, only a modified spill-through type of embankment was tested. The terminology "spill-through" is taken from Liu and Skinner, (1960), but has been modified for use here by the

addition of flexible fibreglass mesh which simulates a protected slope. This type of embankment is very popular in current highway bridge design and so is adequate for general design considerations. Several pictures of the embankment can be seen in the plates in APPENDIX C.

Experimental Procedure

1. Without Constriction

The first series of tests was performed to determine the natural or normal flow characteristics of the channel over the entire range of flows expected for the duration of the work.

As the slope of the channel was fixed in the construction of the model channel, the imposition of a weir discharge necessitated unique equilibrium values of charge and normal depth. Numerous weir discharges were imposed resulting in the same number of equilibrium normal depths and charges. These values permitted rating curves of normal depth and charge versus weir discharge to be obtained (FIGURE B-1, 2). The beginning of bed movement was also observed and the corresponding normal depth and discharge were recorded, (TABLE B-8).

The bed load charge was measured on a volumetric basis. The method used for all runs was to measure the volume of sediment collected in the tail box and to combine this figure with the net deposition or erosion of material from within the experimental channel due to deposition on the inside of the bend or erosion on the outside of the bend. This figure was then compared with the amount of sediment added at the upstream end through the vibrating hopper. The amount of sediment added

was usually in very close agreement to the net amount of sediment discharged so an average of these 2 figures was used. The total amount of sediment transported was then divided by the time of the run to obtain a charge in cubic feet per hour.

As bulking of the sediment was not a problem due to the coarse nature of the bed material, the volumetric bed load charge was converted to parts per 100,000 by weight by means of the water discharge and the unit weight of the bed material.

The general testing procedure that was used for all runs was as follows. The water discharge was set to the desired value by means of the valve at the pump and measured at the weir. Knowing this discharge, the rate of sediment inflow to the channel was set by means of the controls on the variable speed vibrating feeder and hopper. The amount of sediment required was estimated from the work of Cooper and Peterson, (1968). The tail gate was set to give uniform flow conditions over the entire length of the channel. After this was accomplished and equilibrium conditions established by adjusting sediment feed if necessary, profiles were taken by means of a point gauge over the entire length of the channel. These profiles were repeated at least twice to ensure that equilibrium conditions had indeed been established. At the end of each run, the volume of sediment discharged was measured and the rates of sediment discharge were computed by the method outlined above. A similar procedure was used for the tests involving the constricted waterway.

2. With Constriction

The model of the embankment was placed at the end of the straight reach with the center line of the ambankment between Stations 20 and 21. The center line of the embankment was placed at right angles to the mean direction of flow. The placing of the embankment well downstream from the head box allowed the entrance roughness conditions to be dissipated and for fully rough turbulent flow conditions to be established.

The minimum value of R_c (Area constriction ratio) used was 0.5. At high flow with this degree of constriction, the erosive attack on the bed and sides in the vicinity of the embankment was thought to be greater than that which would be allowed in practice. Thus, only one test involving overbank flow was run at this area constriction ratio.

After equilibrium conditions were established in the channel, water surface and bed profiles were taken along the center line of the channel length. Velocity distributions were taken at various points over the width and depth of the channel at the required cross-sections. The measured mean velocity U_m is the average of the numerous point velocities measured on a given cross-section. At each horizontal location on a given cross-section a series of vertical readings were taken, the number depending on the depth of flow. The mean velocity on each vertical on the cross-section was then computed. The maximum value of these mean vertical velocities so computed on a given cross-section was given the symbol U_{max}.

The equipment available did not allow for a continuous recording of depth of scour versus time as the scour hole developed. Visual observations were made in this regard. At the end of each run, bed cross-sections were taken at a number of stations. In the contracted area they were used to compute the scour that had occurred over the period of the run and upstream they were used as a check that the normal bed elevations had remained constant over the duration of the run. The location of the maximum scoured depth was also observed.

CHAPTER IV

PRESENTATION, ANALYSIS AND DISCUSSION OF RESULTS

Complete laboratory test records, giving details of all data collections are on file with the Highways Research Division of the Research Council of Alberta.

A detailed summary of the laboratory data is given in APPENDIX B with an outline of all the laboratory work given in TABLE B-1. The following data are applicable to all experiments:-

Specific gravity of bed material	2.68
Mean energy gradient	.0044 ft./ft.
Mean water temperature	65° F.
Dry bulk weight of bed material	105 lbs./ft.^{3}
Median diameter bed material bulk sample by weight	1.0 millimeter

Flow Conditions in Unconstricted Channel

The calculated weir discharge was compared with that obtained from velocity area computations and a good agreement was found to exist. The higher flows were associated with progressively more bed load movement and associated bed forms. The correspondence between predicted and observed bed load charge was good at low flows with the correspondence becoming progressively worse at higher flows. A detailed tabulation of normal depth, bed load charge, Shields' parameter and bed forms is given in TABLE B-8.

The presence of bed forms led to considerable difficulty in obtaining average depths of flow. The final normal depth obtained was a time-space average.

As the slope of the channel was held constant, it was possible to obtain a rating curve for the normal flow data, (FIGURE B-1). This rating curve was valid for all values of normal depth over the range of discharges used, including overbank flows. The fact that all data are on a well-defined curve attests to the constancy of the data. The fact that the normal depths for the overbank portion of the flow can be extrapolated smoothly from the case of no overbank flow can be explained as follows: the high relative roughness of the flood plain over the range of flows tested made the portion of flow on the flood plain a very small percentage of the total discharge. There is also considerable added resistance due to the interaction of the main channel flow and the overbank flow.

It was expected that a smooth sediment rating curve would also be obtained, but TABLE B-8 indicates that the rate of sediment discharge drops off in the vicinity of the bankfull discharge region and regains the upward trend as the flow increases further. This is also shown in FIGURE B-2.

Bed paving by the coarse fraction of the bed material was observed only at relatively low discharges and low values of the Shields' mobility parameter. If the commonly quoted value of the Shields' number of $0.05 \rightarrow 0.06$ is accepted for the beginning of motion of the bed

material, then observations show that bed paving ceases to be a factor after approximately the D_{65} size by weight is mobile, (TABLE B-8). After the D_{65} is mobile, bed paving was observed to be no longer effective in controlling scour. It was observed visually that as the flow increases past the stage where the D_{50} size begins to move, there is a brief period when the bed is plane, then as more and more particles become mobile the bed forms start to develop. Data for the observed sequence are in TABLE B-8.

The development of bed forms, their associated changes in flow phase and the magnitude of the sediment charge agreed with the diagrams of Cooper and Peterson, (1968).

Due to the large volume of bed material being circulated in the system, a frequent regular check was maintained of the grain size distribution of the sediment being fed into the system. Over the entire series of tests, it was found that there was no noticeable change in the gradation of the sediment.

Flow Conditions in Constricted Channel

The scour data given in APPENDIX B was analysed within the framework of the dimensional analysis developed in CHAPTER II. It was assumed that uniform flow conditions existed in the contracted channel reach. It is realized that this assumption is not strictly satisfied but with it, conventional flow formulas could be used as could the sediment charge curves of Cooper and Peterson, (1968).

The available equipment did not allow the rate of scour to be

obtained. The average testing period was 3 to 4 hours. Visual observations on the time rate of development of scour holes showed that most of the total scour took place within the first 10 to 15 minutes and that effective equilibrium conditions could be accepted after 30 minutes from the start of the test. This is in disagreement with Liu et al (1961) who obtained times in the order of 2 or 3 hours with upstream sediment supply for equilibrium to occur. This rate of development of the scour hole was found to be about the same for the complete range of flows tested.

At very low flows, the fines were immediately washed out of the parent bed material leaving the bed paved and therefore resistant to scour at this flow. An analysis of the bed paving material showed that the D'_{50} size of a scraping of the surface material by weight was approximately equal to 2.5 millimeters which is the D_{80} of the parent bed material. A further analysis, by counting surface stones in a 4-inch grid, PLATE 5, yielded a surface D_{50} of 3.3 millimeters. Due to the difficulties involved in obtaining a 2-grain size thickness of bed material in the scraping for a size by weight measurement and the difficulties involved in counting the smaller particles on a photograph, the effective surface D_{50} of bed paving was estimated to be between the 2 calculated values, i.e., approximately 3.0 millimeters, which is the D_{90} size by weight of the parent material. These values compare favourably with those of Livesey, (1963).

It was observed that the maximum depth of scour occurred in all circumstances within a period of 30 minutes in the model. If the scour

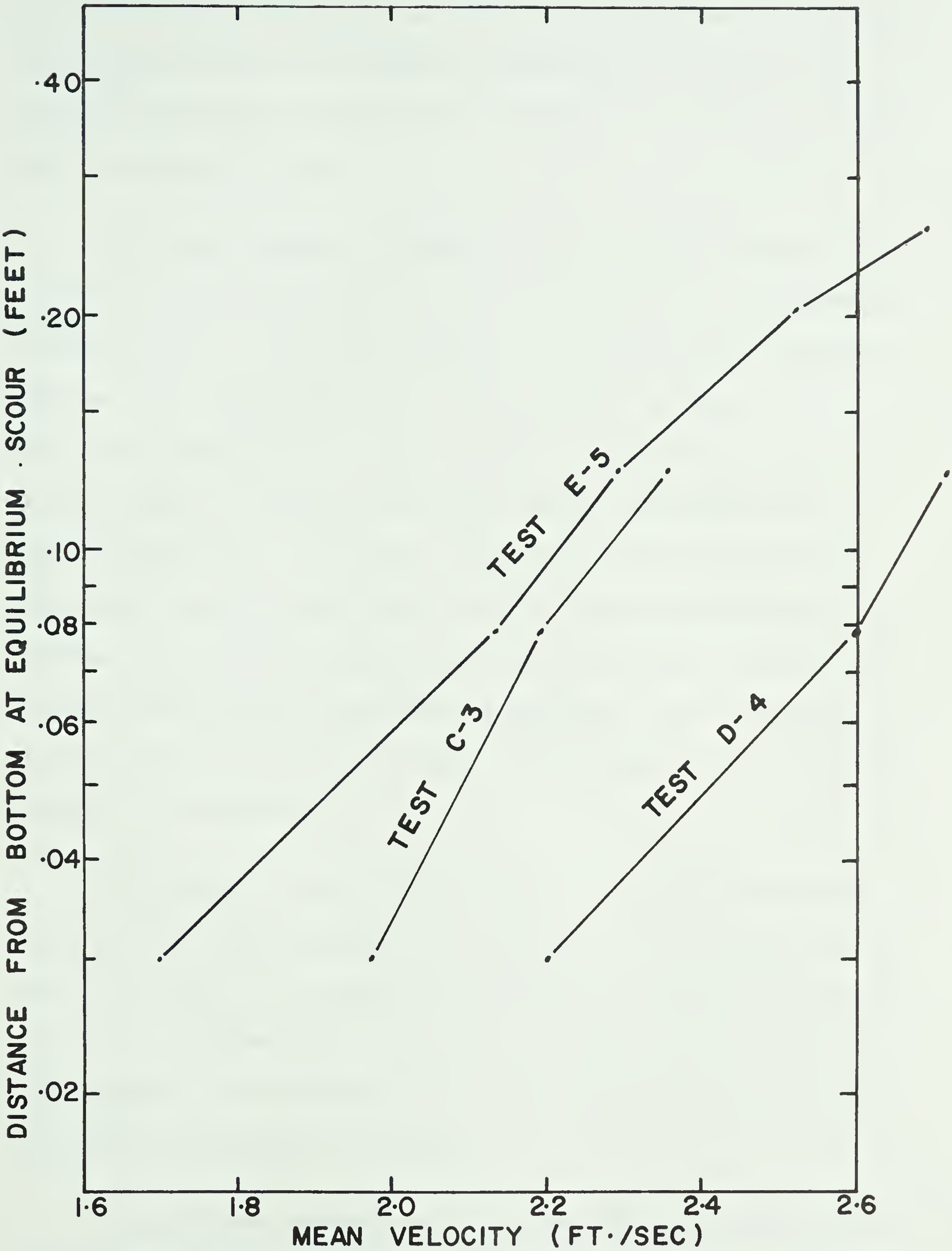


FIGURE 3 VELOCITY DISTRIBUTIONS ON CENTRE-LINE AT STATION 21

time factor can be represented as a function of the square root of the length scale (1:40) as for a purely Froudian model, then the total time for the maximum scouring action to occur in the prototype, would be in the order of 4 hours.

The data presented in TABLES B-9, 10, 11 and 12 indicate that the depth of scour increases as the flow increases for a given amount of constriction. It was also noted that the location of the maximum depth of scour moved downstream as the discharge increased. At low flows, the maximum depth of scour was near the center line of the embankment while at bankfull and overbank stages, the maximum depth occurred downstream of the entire embankment (PLATE 4). The large eddies that were observed in the scour hole caused deposition of longitudinal bars of the fine fraction of the bed material directly downstream of and immediately behind the embankments. This corresponds with the results of Liu et al, (1961). These effects are illustrated in PLATE 3 in APPENDIX C.

The observed velocity profiles plot as a straight line when presented on a semi-logarithmic plot, FIGURE 3. The average velocity given in the data is that obtained by taking an arithmetic average of the numerous measured point velocities, the number of point velocities being dependent on the depth of flow. The original data are on file with the Research Council of Alberta.

Calculations indicated that the mean velocity in the constriction averaged about 6% more than the mean velocity in the unconstricted

TABLE 2

TYPICAL VALUES OF THE VELOCITY RATIO $^{Um}/U_{max}$

FOR CONSTRICTED AND UNCONSTRICTED SECTIONS

Discharge c.f.s.	Station	$^{Um}/U_{max}$
.27	17	.97
	21	.93
.54	17	.91
	21	.97
1.13	17	.87
	21	.93
1.83	17	.85
	21	.94
.26	17	.92
	21	.96
.67	17	.90
	21	.92
1.00	17	.96
	21	.98
2.32	17	.87
	21	.89
2.86	17	.78
	21	.92

Channel for small amounts of constriction. It was found that the amount of velocity increase in the constriction varied directly with the amount of constriction (TABLE B-14). An explanation for this is that equilibrium conditions require an increase in the velocity in the constricted section for the equilibrium movement of bed material as the sediment discharge per foot of width must be higher in the constricted section than in the unconstricted channel. The variation of velocity increase with constriction ratio is shown on FIGURE B-3.

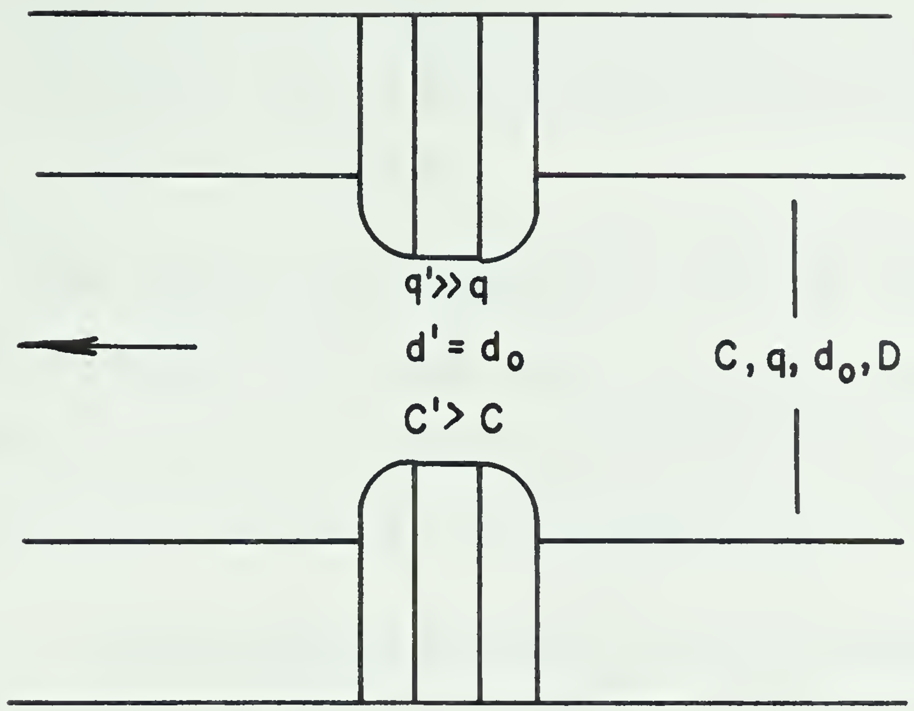
The velocity distribution over the width of the channel in the constricted section was found to approach that of potential flow compared with the usual, curved, velocity distribution found in the upstream channel. The ratio used here as a measure of the shape of the velocity distribution at a given cross-section was defined as U_m/U_{max} where U_m and U_{max} are as defined previously. Values of this ratio given in TABLE 2 show that for a given discharge the velocity distribution in the constricted channel differs in a consistent fashion from the velocity distribution in the upstream, unconstricted channel. The absolute values of this ratio mean little. The smaller the value of the ratio, the more curved the velocity distribution tends to be. The larger the ratio, the closer it is to Unity, the more rectangular is the velocity distribution. With one exception, this ratio was considerably larger at Station 21 than at Station 17, for a given discharge.

The basic scour data are given in TABLES B-9, 10, 11, and 12. APPENDIX A defines the symbols and nomenclature used.

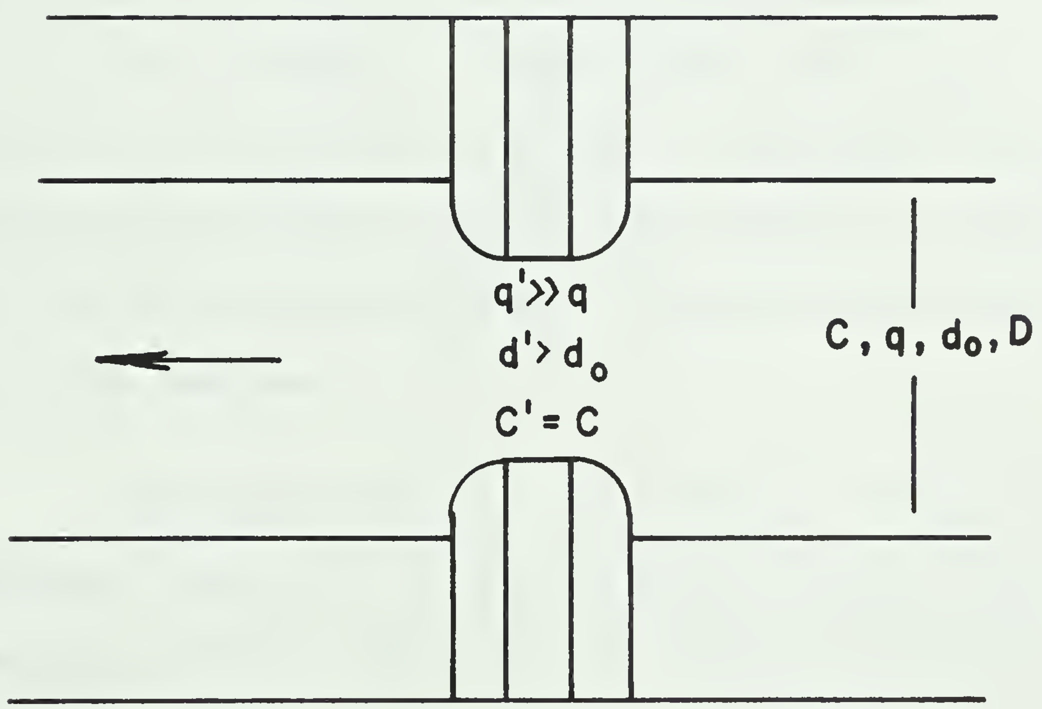
FIGURE A-2 shows the gradation curve for the bed material used in these experiments. The curve indicates that only about 22% of the material is in the range of sizes normally referred to as gravels with the remainder of the material being in the sand size range. Can this material then be used in a model of a gravel bed river? This question first requires the definition of a gravel bed river. It is the author's experience that the term "gravel bed river" can be applied to water-courses whose bed material ranges from large sizes completely within the gravel range to bed materials that are part gravel and part sand. These latter streams usually have their beds paved by the coarse fraction of the bed material especially at low discharges. At higher discharges, these beds are completely mobile. The amount of bed paving and its effect in reducing scour is dependent not only on the absolute size of the coarse fraction but also its relative size and quantity in the total bed material gradation curve. Bed paving was found to be a factor in the experiments reported here. In a model the relative size of the bed material to the geometry of the channel and in particular to the depth of flow has been found to be significant. Also, scaling up of the model to the prototype will yield a greater percentage of the material within the gravel range. These observations then justify the use of this material in the gravel bed model.

Data Analysis

Two methods of scoured depth prediction have been employed in the analysis of the data obtained from these experiments. These methods are:



AT COMMENCEMENT OF FLOW t = 0



AFTER EQUILIBRIUM t = T

FIGURE 4 CHANNEL CONDITIONS BEFORE & AFTER SCOUR

a) Scoured depth from sediment transport formulae.

b) Scoured depth from mean velocity in the channel and constriction geometry.

These methods were developed from the data obtained and then were used to predict the scoured depth that would occur for each test. The predicted values were then compared with those obtained from regime theory and the actual measured values obtained in the experiments.

a) Scoured Depth from Sediment Transport Formulae

Diagrams illustrating the conditions occurring in both the constricted and unconstricted channel at the commencement of flow and after equilibrium has been achieved are shown in FIGURE 4. At the commencement of flow through the constriction, it was assumed that normal flow conditions existed upstream of the embankment. In the embankment itself at this instant, the depth of flow will be equal to the normal depth, the discharge intensity will be much greater than the discharge intensity upstream and as a result, the capacity to carry bed load will be greatly enhanced making the bed load charge much greater than that found upstream.

After equilibrium has been achieved, the depth of flow in the embankment section will be greater than the normal depth upstream, the discharge intensity will be greater than that occurring upstream, and the bed load charge will be equal to that upstream of the embankment. However, the sediment discharge per unit breadth will be greater in the constriction.

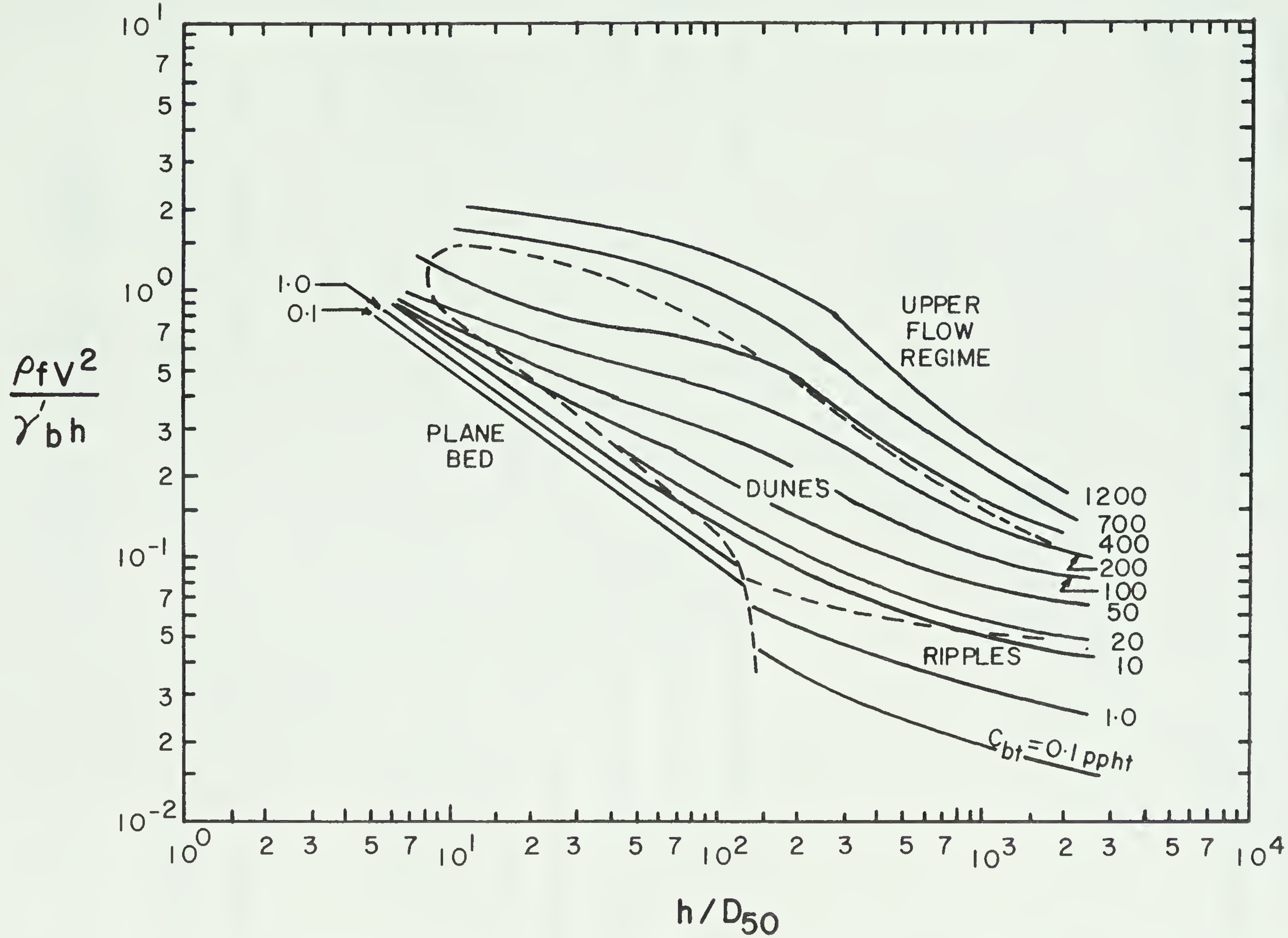


FIGURE 5 BASIC SEDIMENT TRANSPORT DATA COOPER et al (1968)

FIGURE 5 provides a basis for analysis and is an arrangement of the basic sediment transport numerics of Cooper and Peterson, (1968). This diagram has values of a Froude number plotted versus relative depth for different values of bed material charge. The equation for the diagram can be written as,

$$\frac{\rho_f V^2}{\gamma_b' h} = \phi_4 \left(C, \frac{h}{D_{50}}\right) \tag{4}$$

or

$$\frac{\rho_f q^2}{\gamma_b' h^3} = \phi_5 \left(C, \frac{h}{D_{50}}\right) \tag{5}$$

$$\text{since } V = \frac{q}{h}$$

As all the terms on the left hand side of (5) are known from measured information for the unconstricted channel, as is a value of D_{50} for the channel, then a value of the charge C may be predicted from FIGURE 5. If the unit discharge through the contraction q' is now substituted for q in equation (5), and as the charge C will be the same in the constricted and unconstricted sections at equilibrium, then the only remaining variable in equation (5) is the flow depth h, (the scoured depth) which may be solved for in an iterative fashion.

A non-iterative solution for the scoured depth in the contraction may be obtained from the following transformation of equation (5).

$$\frac{\rho_f q^2}{\gamma_b' h^3} \times \left(\frac{h}{D_{50}}\right)^3 = \frac{\rho_f q^2}{\gamma_b' D_{50}{}^3}$$

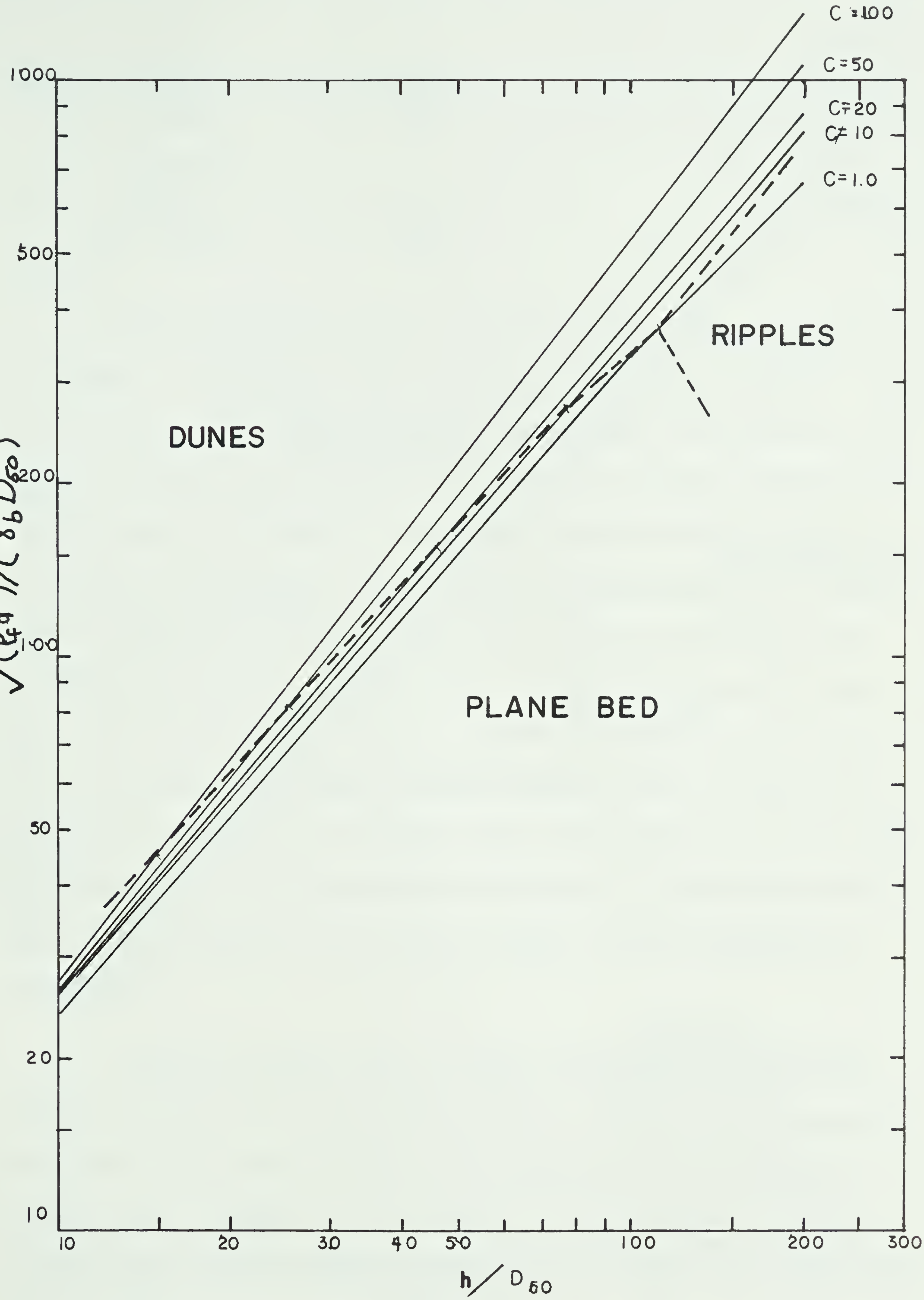


FIGURE 6 TRANSFORMED TRANSPORT DATA

whence $$\frac{\rho_f\ q^2}{\gamma_b'\ D_{50}^{\ 3}} = \phi_6 \quad (C, \frac{h}{D_{50}}) \tag{6}$$

or $$\sqrt{\frac{\rho_f\ q^2}{\gamma_b'\ D_{50}^{\ 3}}} = \phi_7 \quad (C, \frac{h}{D_{50}}) \tag{7}$$

The relationship in (7) is presented with contours of equal C in FIGURE 6 based on the data smoothing lines given in FIGURE 5. The use of this graph to calculate mean scoured depths is as follows: Calculate a value of $\sqrt{(\rho_f\ q^2)/(\gamma_b'\ D_{50}^{\ 3})}$ and h/D_{50} for the unconstricted section and plot on FIGURE 6. Read off a value of bed load charge C, interpolate, if necessary. Next, calculate a value of $\sqrt{(\rho_f\ q'^2)/(\gamma_b' D_{50}^{\ 3})}$ for the constricted section. Using this value and the value of bed load charge obtained in the step above, a value of h/D_{50} may be read off. As a value of D_{50} is known, a value of the mean scoured depth h is thus obtained. The amount of scour below the bed elevation may be obtained by subtracting the normal depth of flow from the scoured depth obtained above. The general use of the charts is illustrated on FIGURE 7. The calculations for each individual test are presented on TABLE B-13.

FIGURE 6 has been used to compare the measured and predicted values of scour in the constricted section. Individual graphs are presented for each series of tests and, as can be seen, the agreement appears good: FIGURES 7, 8, 9, 10 and 11. The only area of discrepancy is that concerned with the magnitude of the bed load charges but the deviations appear to be small.

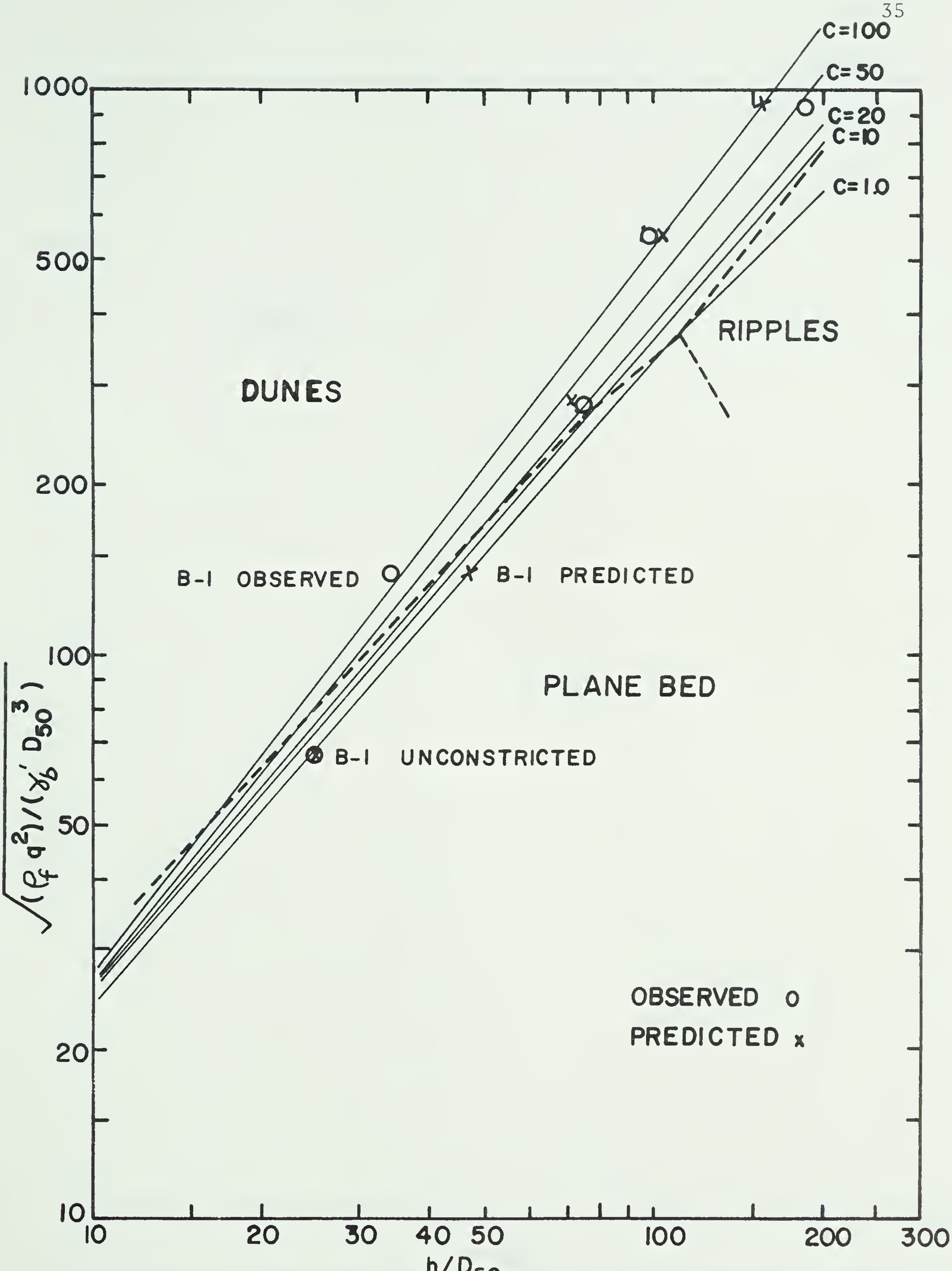


FIGURE 7. OBSERVED AND PREDICTED SCOURED DEPTHS SERIES 'B'

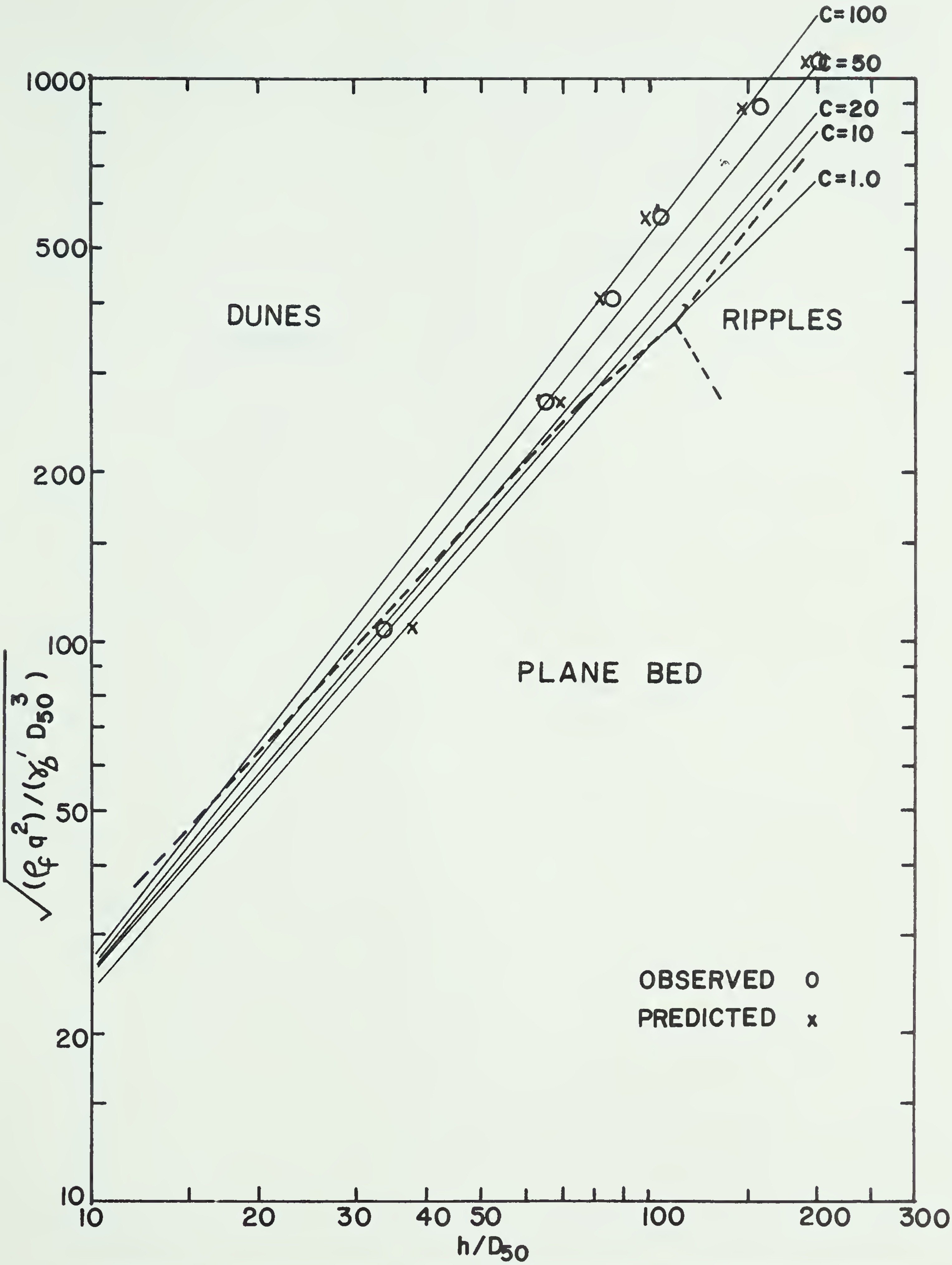


FIGURE 8. OBSERVED AND PREDICTED SCOURED DEPTHS SERIES 'C'

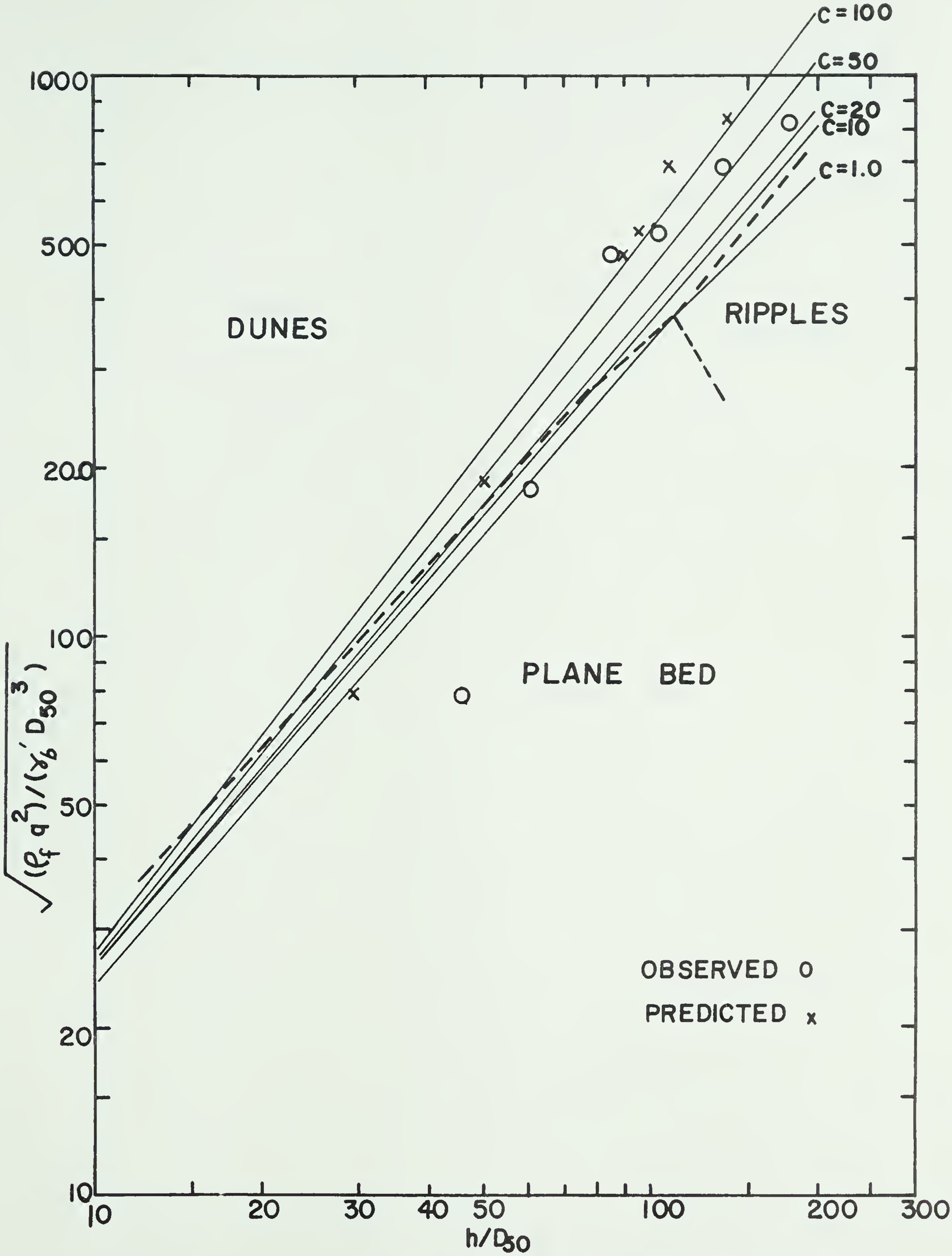


FIGURE 9 OBSERVED AND PREDICTED SCOURED DEPTHS SERIES 'D'

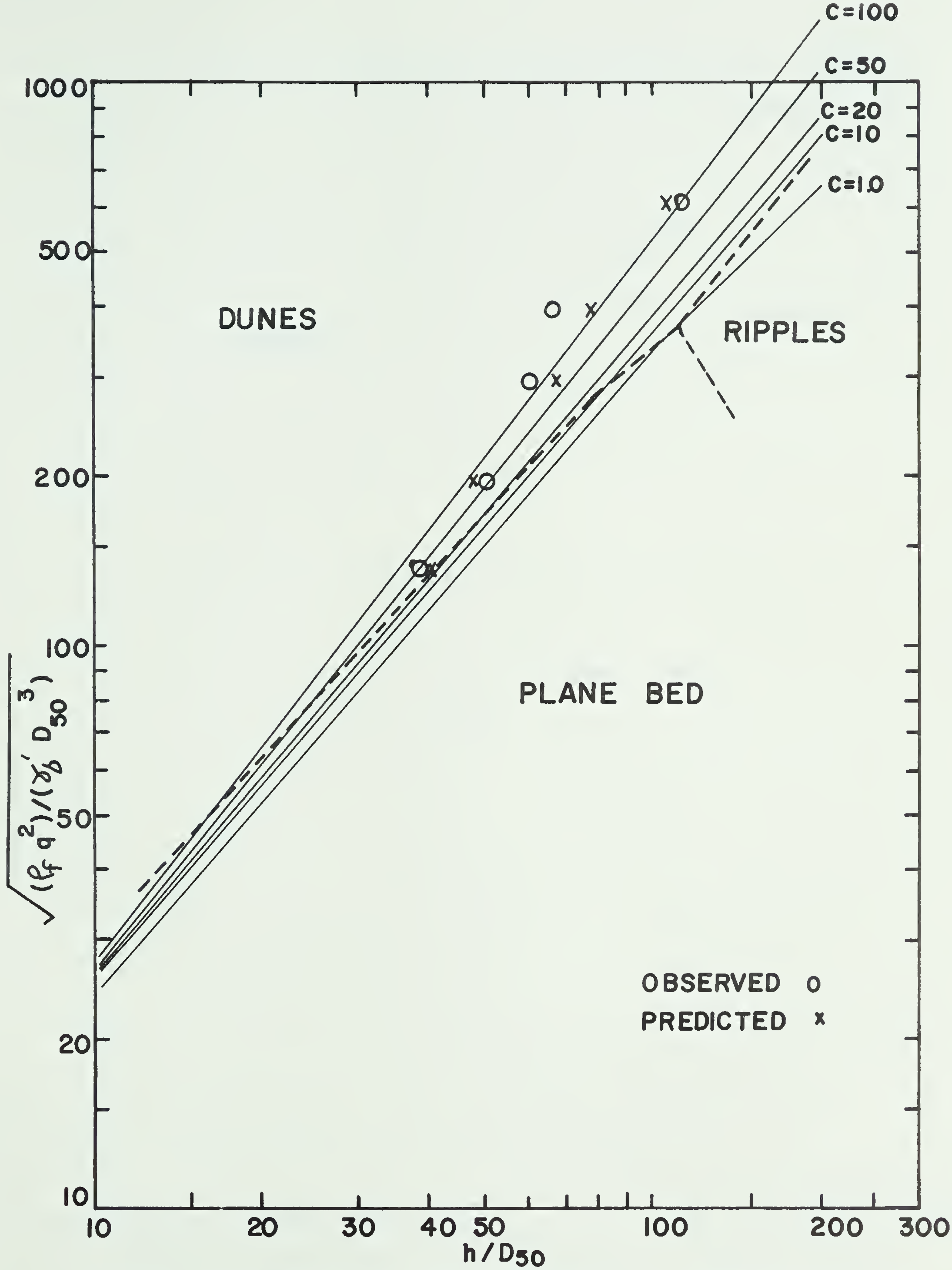


FIGURE 10 OBSERVED AND PREDICTED SCOURED DEPTHS SERIES 'E'

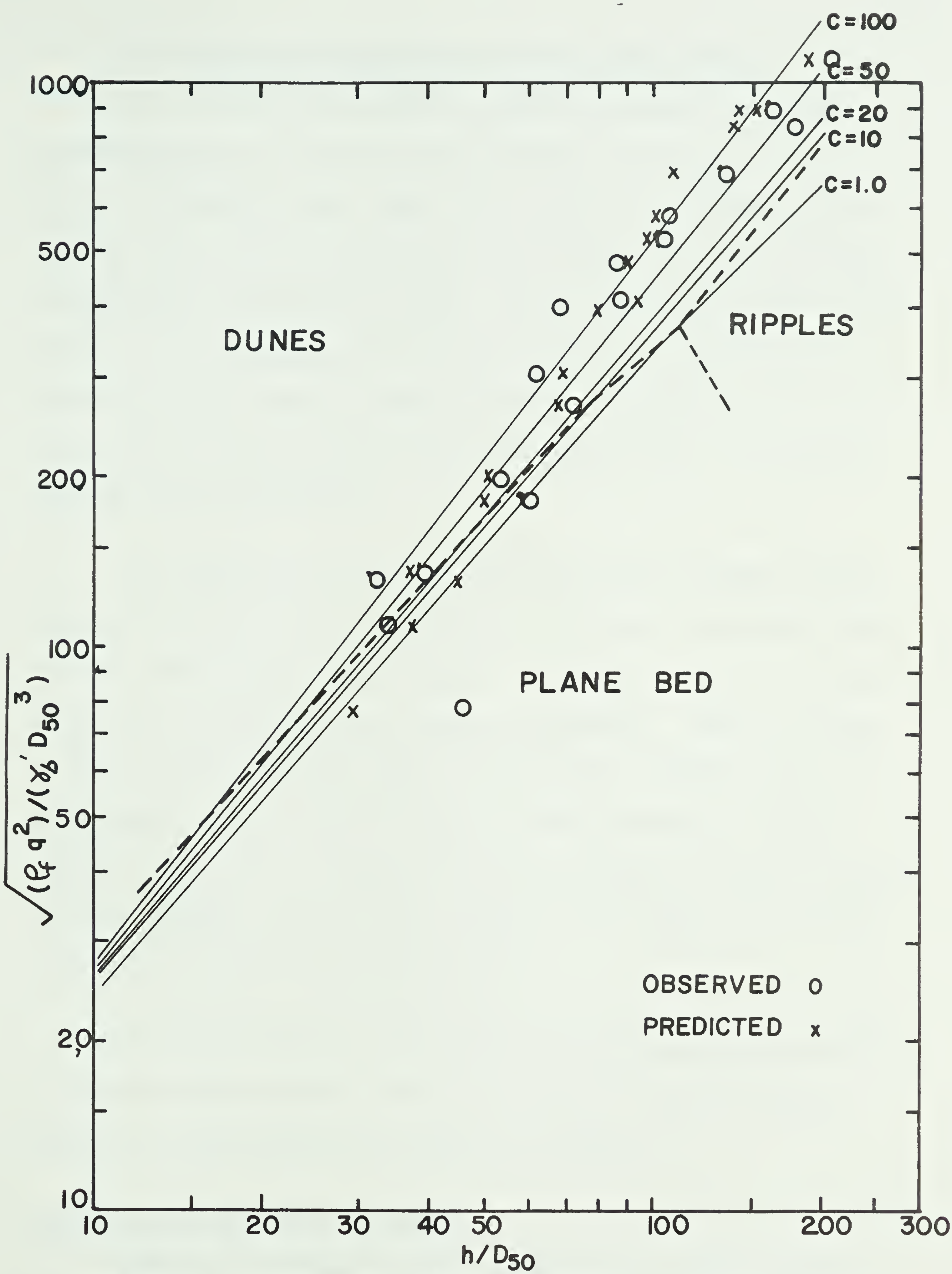


FIGURE II OBSERVED AND PREDICTED SCOURED DEPTHS SERIES 'B,C,D,E'

b) Scoured Depth from Mean Velocity in the Channel

The so-called mean velocity method of average scour depth determination has its origin in the continuity equation, Q = VA. Since the discharge along a short reach of river is constant, it follows that the product of mean velocity and area in the constricted and unconstricted sections must be equal. Thus, as continuity must be satisfied, if the unconstricted flood flow area and the geometry of the constriction were known, it would be possible, by repeated trials, to sketch the flow cross-section in the constriction until the required flow area had been achieved.

The experiments reported here indicate that the mean velocity tends to be greater in the constriction than in the unconstricted channel by a consistent measurable amount. As a result of this, the flow area in the constriction must decrease accordingly. TABLE B-14 gives the data used and the development of the method. COLUMN 9 gives series averages of the velocity increase in the constricted section; COLUMN 11 gives series averages of the area decrease in the constricted section; COLUMN 13 gives series averages of the product of velocity increase and area decrease of each test.

The results indicate that only at extreme constrictions (Series B) does the method appear to break down and give somewhat dubious answers, as illustrated by the results in COLUMN 13.

The Use of the Mean Velocity Method

The use of the mean velocity method of scour depth determination is outlined below. The predicted scoured depth using this method

for each individual test run is calculated and the results shown on TABLE B-15. Also shown are the actual measured values.

The method is based on the fact that as the measured mean velocity in the constricted section as observed in the experiments, has increased by a measurable amount, thus, the flow area in the constricted section must be reduced by a similar amount according to the continuity equation. Thus if the following are known; the geometry of the constriction, the unconstricted flow area, and the amount of the mean velocity ratio increase in the constriction, it is possible to predict the scoured area and hence sketch in the expected scoured section of the constriction. This method allows ample latitude for the designer to select the exact shape of the scoured section based on the specifics of the case being analysed.

The method assumes (a) that the after scour water surface profile through the constriction is the same as it is in the normal unconstricted channel; (b) that the average mean velocity increase in the constriction for a given test series or constriction can be used for calculation purposes for each individual test in that series, and (c) that the average scoured depth below the original bed level can be calculated by assuming that the shape of the scoured cross-section is rectangular of width Wbc.

The steps employed in the method are as follows:

1. From the geometry of the constriction and the flow depth, calculate the constriction area with no scour, i.e. Ac.

2. Calculate the unconstricted flow area and then divide it by the amount that the velocity ratio is increased in the constriction such that the continuity equation is satisfied. (The velocity ratio is based on the results of the experiments reported here. Refer to FIGURE B-3). This is the required flow area of the constriction.
3. Subtract the constriction area Ac from the required flow area calculated in step 2. This gives the required area of scour.
4. Divide the required area of scour (step 3) by the width of the opening Wbc to obtain the mean scour depth.
5. Add the mean scour depth to the normal flow depth d_o to obtain the mean scoured depth.

Comparison with Existing Data

An attempt was made to verify the present author's results with the work of others, notably Laursen et al (1953). Close examination of this work, however, revealed that this was not readily possible as somewhat different flow conditions were used and the data were for local scour near vertical-faced abutments rather than general scour. The most readily accessible general scour information is that of Blench (1969), and was used for comparison purposes (TABLE 3, B-13). Values of the bed factor, F_b, varied upwards from 1.9 depending on the charge according to the relationships given in the original reference. Values of the observed and predicted scoured depths for the regime analysis of Blench and the analyses developed here are presented on TABLE 3 and FIGURE 12.

The average absolute value deviations of the predicted depth

TABLE 3

COMPARISON OF MEASURED AND PREDICTED SCOURED DEPTHS

(1)	(2)	(3)	(4)	(5)
Test	d_{av} Measured Mean Scoured Depth (ft)	Regime Analysis Depth (ft)	Mean Velocity Analysis Depth (ft)	Sediment Transport Analysis Depth (ft)
B-1	.11	.17	.09	.15
B-2	.21	.26	.18	.23
B-3	.38	.36	.27	.33
B-4	.33	.58	.41	.51
C-1	.11	.14	.11	.13
C-2	.22	.27	.19	.22
C-3	.24	.30	.21	.28
C-4	.36	.38	.36	.34
C-5	.55	.58	.75	.50
C-6	.60	.70	.47	.63
D-1	.15	.12	.13	.10
D-2	.20	.18	.18	.17
D-3	.29	.37	.31	.30
D-4	.31	.37	.34	.33
D-5	.45	.45	.69	.36
D-6	.57	.56	.66	.46
E-1	.13	.14	.13	.13
E-2	.18	.18	.17	.17
E-3	.21	.26	d_o = .20	.23
E-4	.23	.32	d_o = .23	.26
E-5	.35	.39	.45	.36

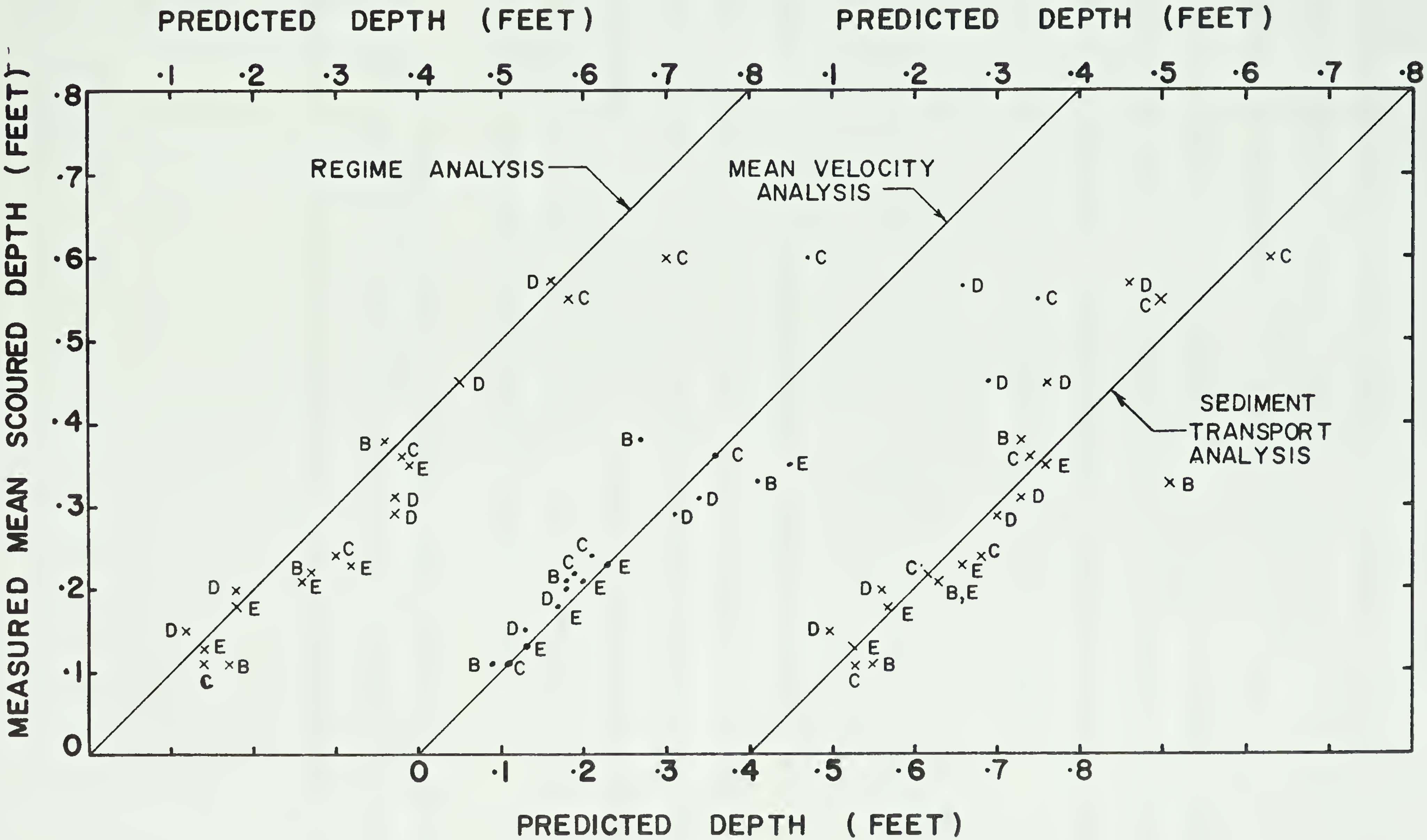


FIGURE 12 COMPARISON OF MEASURED & PREDICTED SCOURED DEPTHS (3 METHODS)

from the measured depth was 20% for the Regime Analysis, 15% for the Mean Velocity Analysis and 13.9% for the Sediment Transport Analysis. When allowance was made for the sign of the deviation all methods on the average overpredicted the scoured depth, the regime analysis by 16.5%, the mean velocity analysis by 1.6% and the sediment transport analysis by 2.6%. This would indicate that the regime analysis yields the most conservative values for design and that mean velocity analysis yields the least conservative values on the average.

Both methods of predicting general scour depths developed here are extremely simple to employ in design. Field surveys and hydrologic investigations are, of course, requisite for both methods. For method (a) Scour depth from sediment transport formulae, it remains only to calculate the two simple ratios of FIGURE 6 to obtain the expected uncontracted sediment charge, and then knowing the geometry of the opening to again use FIGURE 6 to obtain the expected scour depth. For method (b) Scour depth from mean velocity in channel, the previously described steps can be employed with relative ease.

An attempt was made to include a shape factor for the maximum depth of scour, but these results varied widely probably due to local effects and to the effect of the mesh in the constricted test section.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The objective of this research was to determine if there was any rational method or methods of predicting the development and extent of general scour at bridge constrictions on gravel bed rivers. The significant numerics that relate to the general scour problem were defined. These numerics were confirmed with new laboratory data from a relatively large scale hypothetical model of a gravel bed river. The following conclusions are derived and presented on the basis of this quite limited experimental data.

- Bed paving by the coarse fraction of the bed material was found to be a factor only at very low discharges and of insignificant effect at the higher, deeper scouring flows. Where bed paving was a factor, it was determined that the median size of the pavement was equivalent to the D_{90} size by weight of the parent material.

- The location of the maximum depth of scour along the channel centreline varied with the imposed discharge. At low discharges, the maximum depth of scour occurred very near to the centreline of the constriction while at higher discharges, the location of the maximum scoured depth moved progressively downstream. At high overbank flows, the maximum scoured depth was located completely downstream of the constriction.

- Although time rate of scour measurements were not made, visual observations indicated that for the model-prototype relationship of 1:40 used here, the maximum equilibrium depth would be obtained in a prototype river in about 4 hours.

- The average scoured depth can be predicted using the sediment transport analysis and the mean velocity analysis developed here and the regime analysis of Blench. Of these methods, it was determined that the regime analysis on the average yielded the most conservative estimates and the mean velocity analysis yielded the least conservative estimates. Considerably more field and laboratory data will be required to define completely the validity and limits of applicability of the two methods developed here.

Recommendations

It is recommended that:

1. The existing experimental set-up be modified in the following ways to expedite data collection and operational efficiency.
   - Design a new rail system that would not interfere with the flood plain flow and would allow for remote control of the velocity and depth probes at each measuring station. The probe holders, rails and control system should be designed as a single system.
   - Redesign the existing tailgate structure to allow for a more complete control of tailwater level.
2. The author's results be verified.
3. Detailed study be done on the beginning of motion for the gravel mixtures used in this study.

4. Prototype results be obtained to verify the laboratory results.

5. Different bed materials be used to investigate the effect of bed material size on scoured depths and to determine the effects of various gradations on bed paving.

6. Different embankment geometries be investigated to determine its effect on the depth of general scour.

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Appendix A

NOMENCLATURE AND BED MATERIAL ANALYSIS

TABLE A-1

NOMENCLATURE

A	Total measured equilibrium cross-sectional flow area.
A_c	Constriction Area, no scour. $(W_{bc} + 2_{do})d_o$
A_o	Normal flow cross-sectional area. (Calculated).
A_{ob}	1/2 $(A'_o - A_o)$ See FIGURE A-1.
A'_o	Total wetted area, including flood plain.
A_s	Area scoured.
C	Bed load charge, parts per hundred thousand, unless otherwise noted.
C_{bt}	Bed load charge, parts per hundred thousand.
d_o	Normal depth of flow, no constriction.
d'_o	$d_o + \Delta d$.
Δd	Change in water surface elevation on channel centerline, caused by constriction.
d_s	Average depth of scour measured from bed level, $d_s = A_{s/W_{bc}}$ Constricted section. $d_s = A_{s/W_b}$ Unconstricted section.
d_{av}	$d'_o + d_s$ Average scoured depth (deepest cross-section).
d_{max}	Observed maximum depth of scour from water surface to bottom of scour.
D_{50}	Bulk sample particle size of which 50% are smaller by weight.
D_{90}	Bulk sample particle size of which 90% are smaller by weight.
D'_{50}	Surface particle size of which 50% are smaller by weight.
D'_{90}	Surface particle size of which 90% are smaller by weight.

h	General term relating to depth of flow.
Q_w	Weir discharge, cubic feet per second.
q	Discharge intensity, unconstricted section.
q'	Discharge intensity, constricted section.
R_c	A_c / A_o (Area constriction ratio).
U_m	Mean measured velocity, feet per second.
U_{max}	Highest mean-over-vertical velocity, feet per second.
V_m	Q_w / A
V_{mo}	Q_w / A_o
V'_{mo}	Q_w / A'_o
V_{mc}	Q_w / A_c
w_b	Bed width of unconstricted channel.
w_{bc}	Bed width of constricted channel.
α	Shape factor of sediment gradation curve.
β	Shape factor of sediment particles.
ρ_f	Density of water.
ρ_s	Density of sediment.
g	Acceleration due to gravity.
ν	Average kinematic viscosity of fluid.
γ'_b	Unit weight of bed material.
γ	Unit weight of water.
γ_s	Unit weight of sediment.

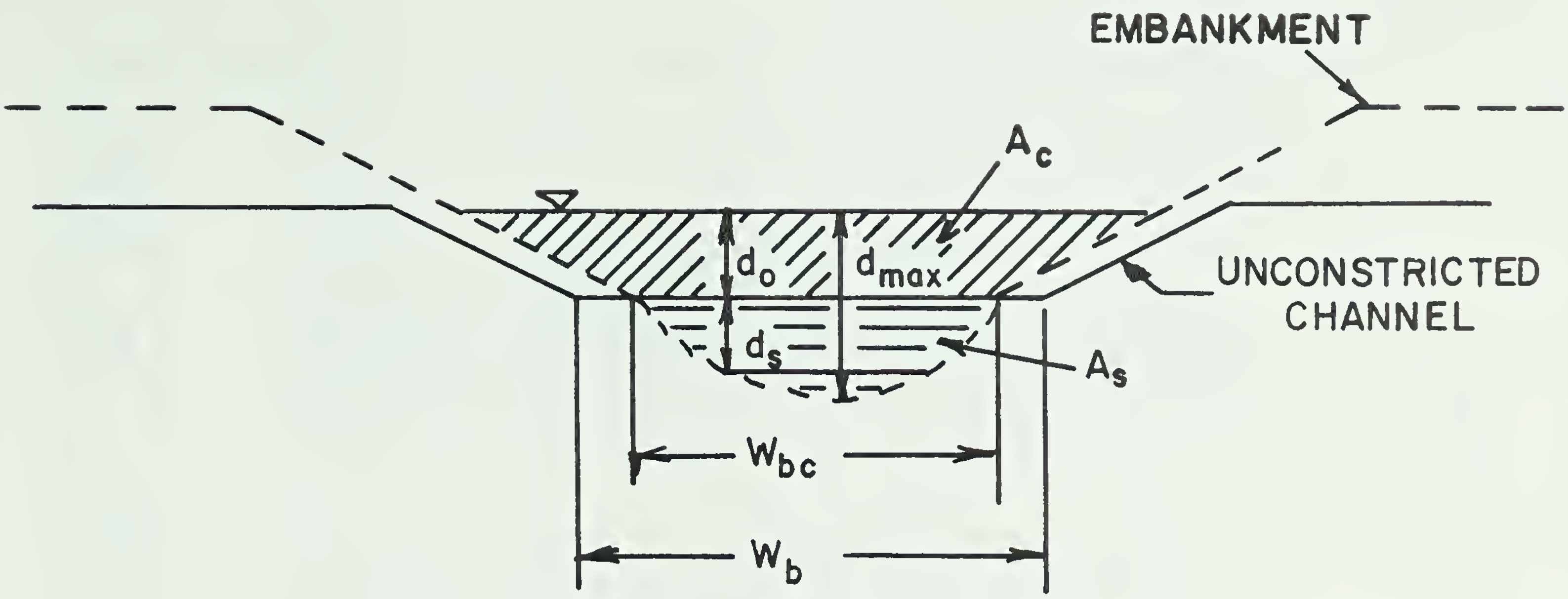

FIGURE A-1a

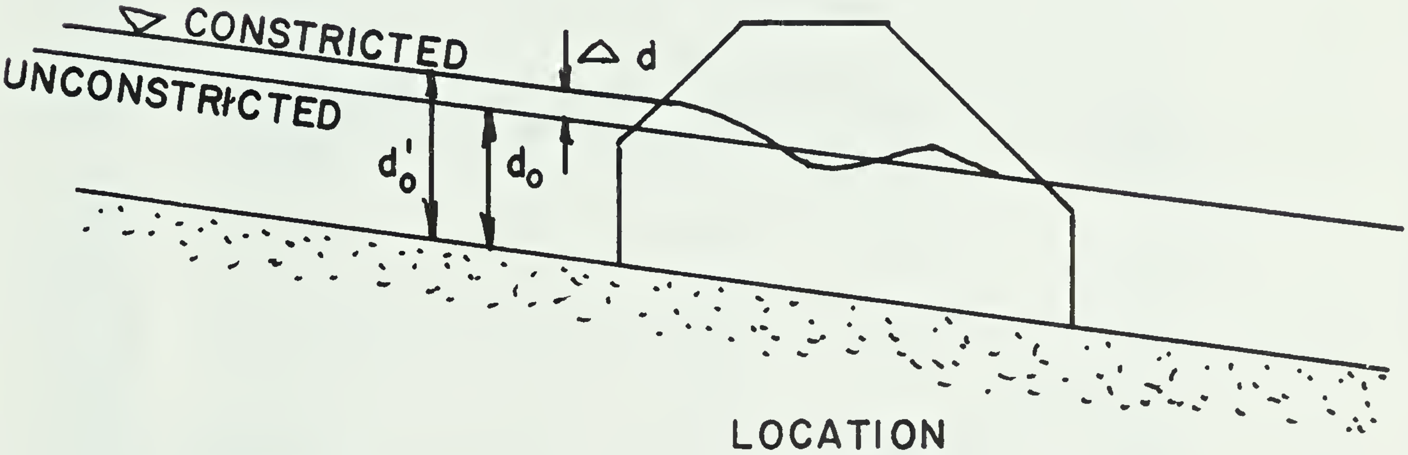

FIGURE A-1b

LOCATION OF EMBANKMENT

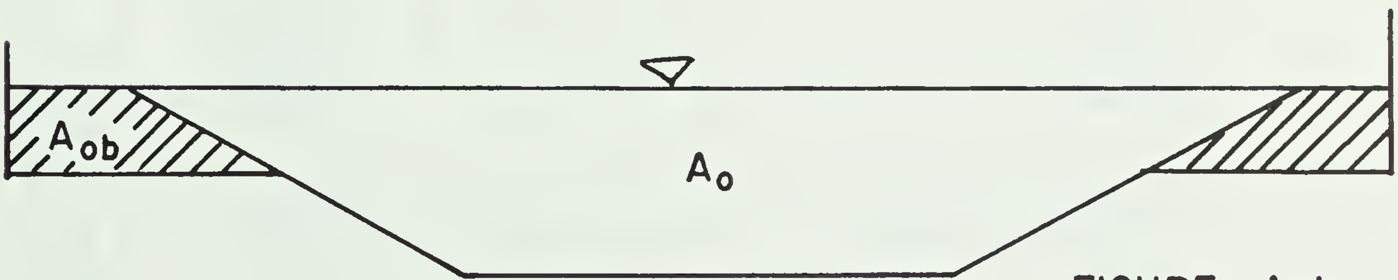

FIGURE A-1c

$A_o' = A_o + 2A_{ob}$

FIGURE A-1 DEFINITION OF SYMBOLS

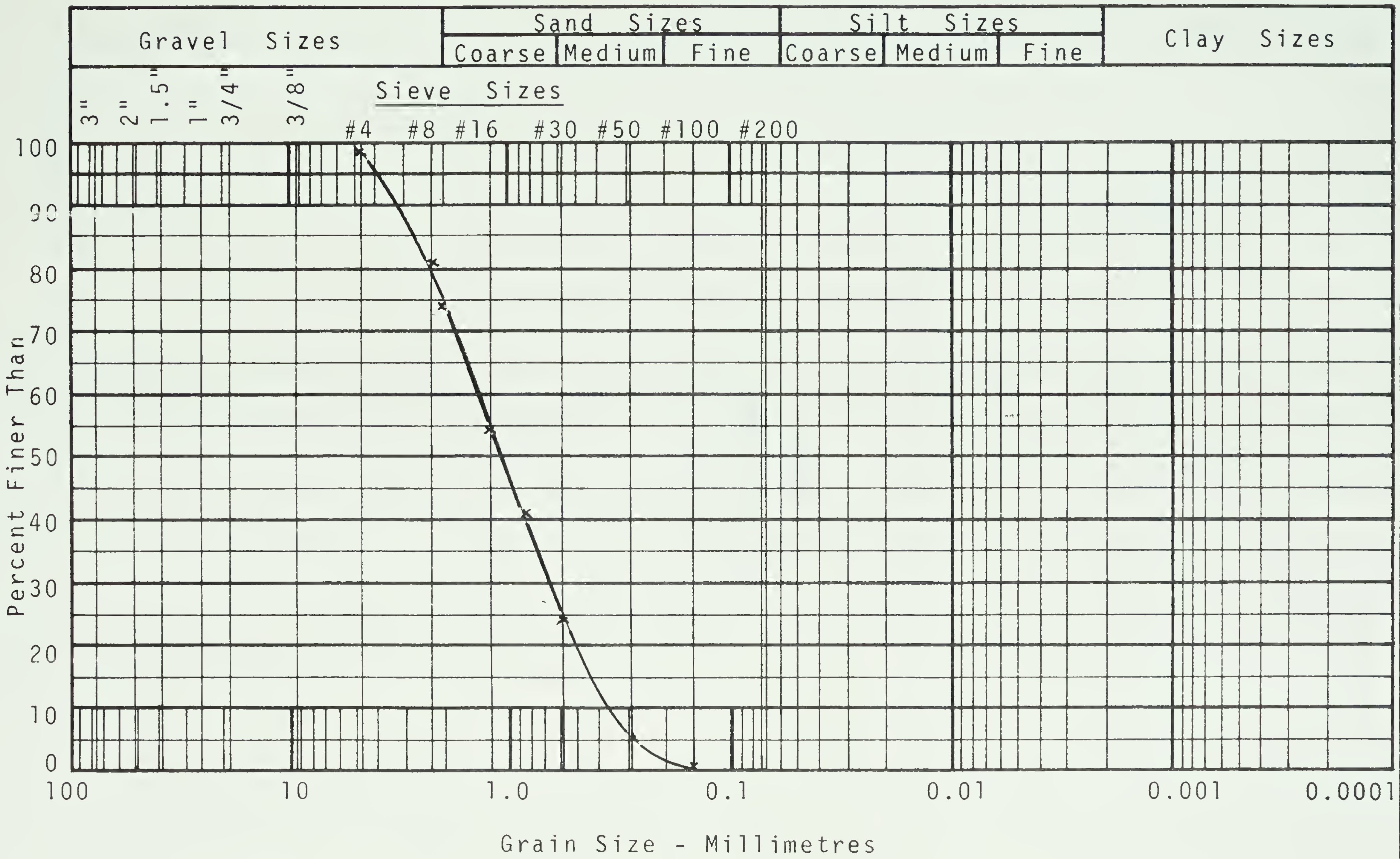

FIGURE A-2 ORIGINAL BED MATERIAL (AVERAGE SPECIFIC GRAVITY = 2·68)

Appendix B

NORMAL FLOW AND CONSTRICTED FLOW DATA

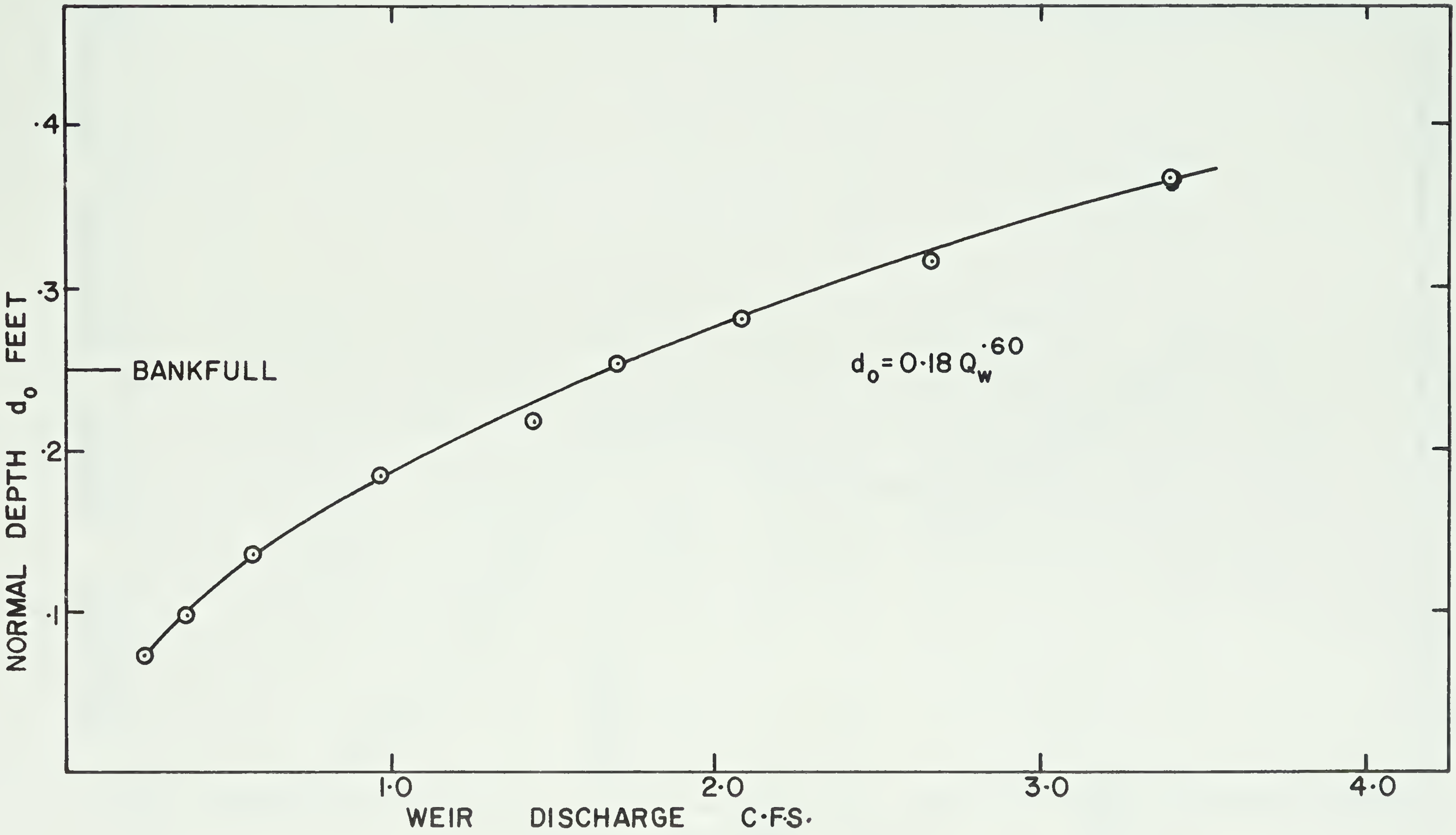

FIGURE B-1 NORMAL DEPTH VS DISCHARGE

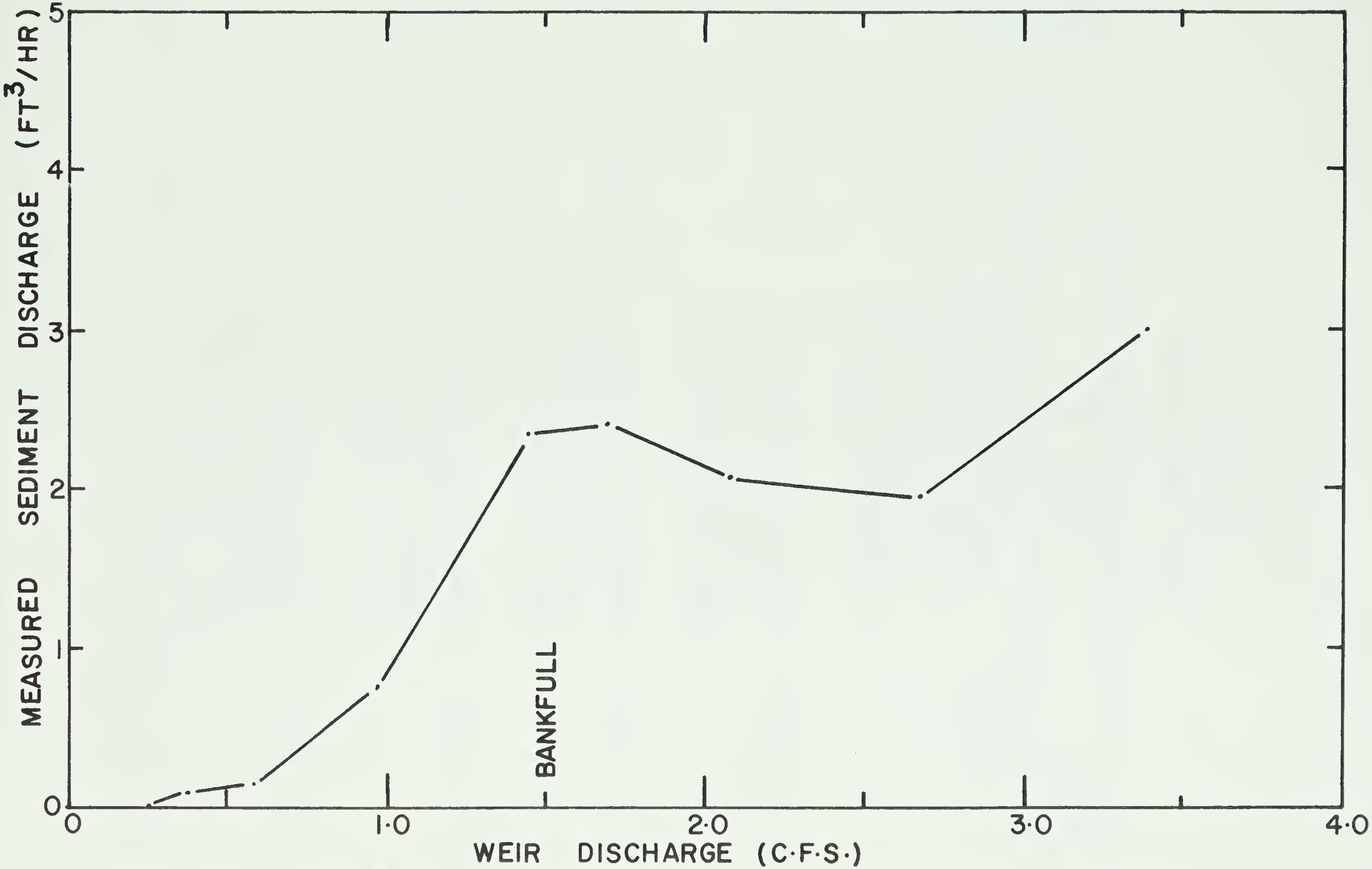

FIGURE B-2 SEDIMENT DISCHARGE VS WEIR DISCHARGE SERIES A

TABLE B-1

OVERALL SUMMARY SHEET

Test Series	Test Runs Included	Main Feature of Series	Details Given in
A	A-1--A-9	To establish flow and transport conditions in experimental channel over a range of discharges (uniform flow, no constrictions, bed width 36")	Table B-2, B-7, 8 Figure B-1
B	B-1--B-4	To measure scour patterns and velocity distributions in a 17" bed width constriction over a range of discharges.	Table B-3, 9
C	C-1--C-6	To measure scour patterns and velocity distributions in a 20" bed width constriction over a range of discharges.	Table B-4, 10
D	D-1--D-6	To measure scour patterns and velocity distributions in a 23" bed width constriction over a range of discharges.	Table B-5, 11
E	E-1--E-5	To measure scour patterns and velocity distributions in a 32" bed width constriction over a range of discharges.	Table B-6, 12

TABLE B-2

SERIES SUMMARY SHEET

TEST SERIES A
(no constriction - bed width 36")

Test Run	Date	Discharge (cfs)	Flow Depth (ft)	Principal Measurements
A-1	Nov. 4/69	0.25	.072	Velocity distribution at Stns. 13, 17, 20, 21, 23; bed transport rate; surface grain size distribution.
A-2	July 22/69	.37	.097	Velocity distribution at Stns. 14, 21, 28, 32; bed transport rate.
A-3	July 23/69	.58	.134	Velocity distribution at Stns. 14, 21, 28, 32; bed transport rate.
A-4	July 28/69	.97	.185	Bed transport rate.
A-5	Nov. 5/69	1.45	.216	Bed transport rate; velocity distribution at Stns. 13, 17, 20, 21, 23; surface grain size distribution.
A-6	July 28/69	1.70	.252	Bed transport rate; velocity distribution at Stn. 21.
A-7	Nov. 6/69	2.08	.280	Velocity distribution at Stns. 13, 17, 20, 23; bed transport rate; surface grain size distribution.
A-8	Nov. 10/69	2.63	.315	Velocity distribution at Stns. 13, 17, 19, 21; bed transport rate; surface grain size distribution.
A-9	Nov. 10/69	3.41	.368	Velocity distribution at Stns. 13, 17, 20, 23; bed transport rate; surface grain size distribution.

TABLE B-3

SERIES SUMMARY SHEET

TEST SERIES B
(constricted bed width - 17")

Test Run	Date	Discharge (cfs)	Flow Depth (ft)	Principal Measurements
B-1	Sept. 8/69	.27	.08	Velocity distribution at Stns.17, 20, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
B-2	Sept. 9/69	.54	.12	Velocity distribution at Stns.17, 20, 21, 23; bed transport rate; bed cross-section.
B-3	Sept.10/69	1.13	.19	Velocity distribution at Stns.17, 20, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
B-4	Sept.15/69	1.83	.26	Velocity distribution at Stns.17, 20, 23; bed transport rate; bed cross-section; surface grain size distribution.

TABLE B-4

SERIES SUMMARY SHEET

TEST SERIES C
(constricted bed width - 20")

Test Run	Date	Discharge (cfs)	Flow Depth (ft)	Principal Measurements
C-1	Aug. 25/69	.26	.08	Velocity distribution at Stns.17, 19, 20, 21, 24; bed transport rate; bed cross-section; surface grain size distribution.
C-2	Aug. 26/69	.67	.14	Velocity distribution at Stns.17, 20, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
C-3	Aug. 27/69	1.00	.18	Velocity distribution at Stns.17, 20, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
C-4	Aug. 28/69	1.51	.23	Velocity distribution at Stns.17, 19, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
C-5	Sept. 3/69	2.32	.30	Velocity distribution at Stns.17, 20, 21, 23; bed transport rate; bed cross-section.
C-6	Sept. 6/69	2.86	.34	Velocity distribution at Stns.17, 20, 27; bed transport rate; bed cross-section.

TABLE B-5

SERIES SUMMARY SHEET

TEST SERIES D
(constricted bed width - 23")

Test Run	Date	Discharge (cfs)	Flow Depth (ft)	Principal Measurements
D-1	Sept.21/69	.21	.07	Velocity distribution at Stns. 17, 21, 24; bed transport rate; bed cross-section; surface grain size distribution
D-2	Oct. 22/69	.51	.12	Velocity distribution at Stns. 13, 17, 20, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
D-3	Oct. 23/69	1.34	.22	Velocity distribution at Stns. 13, 17, 20, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
D-4	Oct. 27/69	1.48	.23	Velocity distribution at Stns. 13, 17, 20, 21, 23; bed transport rate; bed cross-section; surface grain size distribution.
D-5	Oct. 29/69	2.08	.28	Velocity distribution at Stns. 13, 17, 10, 20; bed transport rate; bed cross-section; surface grain size distribution.
D-6	Oct. 31/69	2.44	.31	Velocity distribution at Stns. 13, 17, 20, 21, 23, 25; bed transport rate; bed cross-section; surface grain size distribution.

TABLE B-6

SERIES SUMMARY SHEET

TEST SERIES E
(constricted bed width - 32")

Test Run	Date	Discharge (cfs)	Flow Depth (ft)	Principal Measurements
E-1	Aug. 7/69	.51	.12	Velocity distribution at Stns. 17, 20, 21, 23; bed transport rate; bed cross-section.
E-2	Aug. 8/69	.75	.15	Velocity distribution at Stns. 17, 20, 21, 23; bed transport rate; bed cross-section.
E-3	Aug. 8/69	1.15	.20	Velocity distribution at Stns. 17, 20, 21, 23; bed transport rate; bed cross-section.
E-4	Aug. 8/69	1.53	.23	Velocity distribution at Stns. 17, 20, 21, 23; bed transport rate; bed cross-section.
E-5	Aug.11/69	2.48	.31	Velocity distribution at Stns. 17, 20, 21, 23; bed transport rate; bed cross-section.

	TEST A-1			TEST A-2			TEST A-3			TEST A-4			TEST A-5			TEST A-6			TEST A-7			TEST A-8			TEST A-9		
	Q_w = .25 d_o = .072			Q_w = .37 d_o = .097			Q_w = .58 d_o = .134			Q_w = .97 d_o = .185			Q_w = 1.45 d_o = .216			Q_w = 1.70 d_o = .252			Q_w = 2.08 d_o = .280			Q_w = 2.63 d_o = .315			Q_w = 3.41 d_o = .368		
1 STN.	2 Water Elev.*	3 Bed Elev.*	4 U_m (ft/sec)	5 Water Elev.	6 Bed Elev.	7 U_m (ft/sec)	8 Water Elev.	9 Bed Elev.	10 U_m (ft/sec)	11 Water Elev.	12 Bed Elev.	13 U_m (ft/sec)	14 Water Elev.	15 Bed Elev.	16 U_m (ft/sec)	17 Water Elev.	18 Bed Elev.	19 U_m (ft/sec)	20 Water Elev.	21 Bed Elev.	22 U_m (ft/sec)	23 Water Elev.	24 Bed Elev.	25 U_m (ft/sec)	26 Water Elev.	27 Bed Elev.	28 U_m (ft/sec)
6				9.606	9.507		9.624	9.445		9.657	9.396					9.730	9.415										
9				9.588	9.491		9.607			9.634	9.450					9.705	9.457										
13	9.531	9.437	1.18	9.570	9.470	1.18	9.595	9.435	1.58	9.618	9.453	1.64	9.690	9.460	2.11	9.700	9.450		9.735	9.464	2.19	9.775	9.474	2.18	9.796	9.435	2.22
17	9.517	9.405	1.07	9.561	9.462		9.586			9.611	9.427		9.655	9.442	2.04	9.690	9.423		9.725	9.441	2.18	9.756	9.433	2.25	9.798	9.415	2.27
19	9.515	9.411											9.675	9.458					9.720	9.448		9.745	9.456	2.19	9.785	9.411	
20	9.503	9.419	1.05										9.676	9.423	2.06				9.710	9.438	2.00	9.743	9.424		9.782	9.405	2.37
21	9.497	9.431	1.04	9.534	9.442	1.24	9.567		1.41	9.600	9.413	1.73	9.664	9.427	1.99	9.667	9.403	2.07	9.705	9.395		9.734	9.436	2.19	9.777	9.423	
22	9.490	9.426											9.656						9.708	9.438		9.730			9.770	9.412	
23	9.486	9.422	1.05										9.650	9.412	1.71				9.702	9.433	2.06	9.729	9.440		9.772	9.397	2.29
24	9.485	9.421											9.637						9.703	9.441		9.719			9.771	9.392	
25	9.485	9.413		9.522	9.423		9.553	9.411		9.585	9.395		9.641	9.409		9.658	9.383		9.705	9.430		9.720	9.404		9.769	9.413	
27	9.485	9.362											9.639	9.425					9.699	-		9.720	9.410		9.767	9.409	
29	9.486	9.359		9.505	9.409	1.26	9.540		1.43	9.575	9.371	1.58	9.628			9.643	9.404								9.765	9.408	
33				9.486	9.385	1.26	9.520	9.366	1.50	9.562	9.371					9.623	9.425										
37				9.463	9.370		9.496			9.540	9.407					9.603	9.443										
41				9.445	9.350		9.474	9.364		9.522	9.354					9.573	9.280										
45				9.429	9.333		9.455			9.505	9.303					9.557	9.293										

* Water and bed elevations are in feet referred to datum of page 12.

Mean Channel Slope .0044 Feet/Feet

TABLE B-7 DATA ON NORMAL FLOW CONDITIONS (BED WIDTH 36" NO CONSTRICTION)

TABLE B-8

BED MATERIAL MOVEMENT IN UNCONSTRICTED CHANNEL

Qw (Ft^3/sec.)	d_o (Feet)	$\frac{\gamma d_o S}{(\gamma_s - \gamma) D_{50}}$	$\frac{\gamma d_o S}{(\gamma_s - \gamma) D_{65}}$	C (Measured) Ft^3/hr.	C (Calc) Cooper Peterson Ft^3/hr.	Comments
.25	.072	.058	.038	0	.0005	1 mm. material mobile, bed paving
.37	.097	.078	.052	.1	.078	1 mm. and larger mobile, bed paving
.58	.134	.107	.072	.16	.34	All but largest sizes mobile; plane bed, no paving
.97	.185	.148	.098	.77	.81	All sizes mobile; plane bed, no paving
1.45	.216	.173	.116	2.35	3.13	Bed forms 15" spacing 1/4" high
1.70	.252	.201	.134	2.40	5.12	Bed forms 20" spacing 3/8" high
2.08	.280	.224	.150	2.08	4.40	Bed forms 30" spacing 1" high
2.68	.315	.252	.166	1.95	5.23	Bed forms 49" spacing 2" high
3.40	.368	.294	.192	3.0	6.75	Bed forms 48" spacing 2" high

(1) TEST NO.	(2) STATION	(3) Q_w (cfs)	(4) A_o (ft^2)	(5) d_o (ft)	(6) V_{mo} (ft/sec)	(7) A_c (ft^2)	(8) V_{mc} (ft/sec)	(9) w_{bc} (ft)	(10) A'_o (ft^2)	(11) V'_{mo} (ft/sec)	(12) C (ft^3/hr)	(13) D'_{50} (mm)	(14) D'_{90} (mm)	(15) $\triangle d$ (ft)	(16) d'_o (ft)	(17) A (ft^2)	(18) A_s (ft^2)	(19) d_s (ft)	(20) d_{av} (ft)	(21) d_{max} (ft)	(22) V_m (ft/sec)	(23) U_m (ft/sec)	(24) U_{max} (ft/sec)	(25) LENGTH OF RUN (mins)
B-1	17	.27	.26	.08	1.04	.13	2.08	1.42	-	-	0.0	3.0	4.7	.010	.09	.19	-.10	-.03	.06	-	1.40	1.09	1.13	370
	20											2.3	4.3	.010	.09	.18	.03	.02	.11	.19	1.54	1.49	1.61	
	21											2.5	4.5	-.001	.08	.16	.04	.03	.11	-	1.65	1.40	1.51	
	22											2.0	4.3	.000	.08	.18	-.08	-.03	.06	-	1.47	1.17	1.55	
B-2	17	.54	.40	.12	1.35	.21	2.62	1.42	-	-	0.47	2.0	4.0	.006	.13	.37	-.05	-.02	.11	-	1.46	1.48	1.62	325
	20													.010	.13	.32	.10	.07	.21	.38	1.66	1.75	1.84	
	21													.011	.14	.38	.15	.11	.24	-	1.42	1.76	1.81	
	23													-.008	.12	.40	.03	.01	.12	-	1.34	1.62	1.87	
B-3	17	1.13	.66	.19	1.71	.35	3.23	1.42	-	-	1.67	1.8	4.0	.040	.23	.59	-.22	-.07	.16	-	1.91	1.91	2.20	215
	20													.012	.21	.53	.16	.11	.32	-	2.12	2.44	2.56	
	21													.000	.19	.62	.27	.19	.38	.49	1.83	2.29	2.48	
	23													.020	.21	.75	.01	.00	.22	-	1.51	1.71	2.34	
B-4	17	1.83	.91	.26	2.01	.50	3.65	1.42	.98	1.86	1.45	1.5	3.8	.013	.27	.91	-.14	.05	.32	-	2.00	1.85	2.19	152
	20											1.0	3.0	.013	.27	.74	.09	.06	.33	-	2.48	2.32	2.47	
	21											1.1	-	.010	.28	.71	.07	.05	.33	-	2.58	-	-	
	23											1.0	3.0	-.010	.25	1.90	1.11	.37	.62	.76	.96	1.85	2.20	

TABLE B-9 BASIC SCOURED DATA TEST SERIES B CONSTRICTED BED WIDTH 17"

(1) TEST NO.	(2) STATION	(3) Q_w (cfs)	(4) A_o (ft^2)	(5) d_o (ft)	(6) V_{mo} (ft/sec)	(7) A_c (ft^2)	(8) V_{mc} (ft/sec)	(9) w_{bc} (ft)	(10) A'_o (ft^2)	(11) V'_{mo} (ft/sec)	(12) C (ft^3/hr)	(13) D_{50} (mm)	(14) D_{90} (mm)	(15) Δd (ft)	(16) d'_o (ft)	(17) A (ft^2)	(18) A_s (ft^2)	(19) d_s (ft)	(20) d_{ov} (ft)	(21) d_{max} (ft)	(22) V_m (ft/sec)	(23) U_m (ft/sec)	(24) U_{max} (ft/sec)	(25) LENGTH OF RUN (mins)
C-1	17	.26	.25	.08	1.03	.15	1.78	1.67	-	-	0	2.1	4.0	-.008	.07	.24	.01	.00	.08	-	1.11	1.23	1.33	510
	20											2.3	4.3	.000	.08	.20	.06	.03	.11	.18	1.29	1.40	1.51	
	21											2.3	4.3	-.016	.06	.17	.06	.03	.10	-	1.52	1.40	1.46	
	24											1.8	4.0	-.008	.07	.25	.03	.01	.08	-	1.02	0.99	1.34	
C-2	17	.67	.46	.14	1.45	.28	2.44	1.67	-	-	.58	2.0	4.0	.006	.15	.43	-.06	-.02	.13	-	1.57	1.57	1.73	310
	20											-	-	.004	.15	.42	.13	.08	.22	.33	1.60	1.65	1.76	
	21											-	-	-.001	.14	.40	.12	.07	.21	-	1.69	1.75	1.91	
	23											-	-	.000	.14	.43	-.04	-.12	.13	-	1.57	1.55	1.83	
C-3	17	1.00	.61	.18	1.65	.37	2.47	1.67	-	-	1.76	2.3	4.8	032	.21	.52	-.20	-.07	.15	-	1.91	1.94	2.00	255
	20											1.9	4.5	.014	.19	.47	.08	.05	.24	.40	2.11	2.10	2.17	
	21											1.7	3.3	.005	.19	.56	.18	.11	.30	-	1.78	2.15	2.23	
	23											2.7	4.0	.007	.19	.55	-.08	-.03	.16	-	1.81	1.98	2.20	
C-4	17	1.51	.78	.23	1.90	.49	3.09	1.67	-	-	2.52	1.8	4.6	.042	.27	.85	-.11	-.04	.23	-	1.78	1.97	2.40	190
	20											1.4	3.8	.014	.24	.69	.16	.10	.34	-	2.20	2.55	2.69	
	21											1.3	3.6	.012	.24	.71	.19	.11	.36	.51	2.13	-	-	
	23											2.1	4.6	.006	.24	.84	.02	-.01	.23	-	1.80	1.96	2.44	
C-5	17	2.32	1.07	.30	2.16	.67	3.44	1.67	1.45	1.60	2.25	1.0	3.5	.002	.30	1.72	.26	.09	.38	-	1.35	1.90	2.18	175
	20											-	-	-.022	.28	1.01	.40	.24	.52	-	2.31	2.39	2.50	
	21											-	-	-.022	.27	1.09	.48	.29	.55	-	2.14	2.49	2.81	
	23											-	-	.000	.25	2.35	.90	.30	.55	.80	.99	2.07	2.80	
C-6	17	2.86	1.24	.34	2.30	.79	3.61	1.67	1.93	1.48	2.50	1.0	3.5	.017	.36	1.25	-.48	.16	.20	-	2.28	1.67	2.12	100
	20											-	-	.005	.34	1.38	.57	.35	.69	-	2.07	2.17	2.37	
	21											-	-	-.022	.32	1.20	.48	.29	.60	-	2.38	-	-	
	23											-	-	-.035	.30	3.07	.72	.24	.54	.97	0.93	1.93	2.37	

TABLE 8-10 BASIC SCOURED DATA TEST SERIES C CONSTRICTED BED WIDTH 20"

(1) TEST NO.	(2) STATION	(3) Q_w (cfs)	(4) A_o (ft^2)	(5) d_o (ft)	(6) V_{mo} (ft/sec)	(7) A_c (ft^2)	(8) V_{mc} (ft/sec)	(9) w_{bc} (ft)	(10) A'_o (ft^2)	(11) V'_{mo} (ft/sec)	(12) C (ft^3/hr)	(13) D'_{50} (mm)	(14) D'_{90} (mm)	(15) $\triangle d$ (ft)	(16) d'_o (ft)	(17) A (ft^2)	(18) A_s (ft^2)	(19) d_s (ft)	(20) d_{av} (ft)	(21) d_{max} (ft)	(22) V_m (ft/sec)	(23) U_m (ft/sec)	(24) U_{max} (ft/sec)	(25) LENGTH OF RUN (mins)
D-1	17	.21	.22	.07	.94	.15	1.44	1.92	-	-	0	2.0	4.0	-.015	.06	.28	.11	.04	.09	-	.74	1.29	1.58	275
	20											2.5	4.3	-.014	.06	.29	.18	.09	.15	.15	.73	-	-	
	21											2.5	4.3	-.010	.06	.29	.18	.09	.16	-	.72	1.25	1.40	
	23											1.8	3.7	-.012	.06	.46	.22	.07	.13	-	.46	.90	1.14	
D-2	17	.51	.39	.12	1.31	.26	1.97	1.92	-	-	.27	2.5	4.2	-.008	.11	.40	.02	.01	.12	-	1.28	1.53	1.65	535
	20											2.1	4.3	-.005	.12	.42	.17	.09	.20	.25	1.22	1.54	1.58	
	21											2.1	4.3	-.005	.12	.41	.16	.08	.20	-	1.25	1.45	1.61	
	23											-	-	-.013	.11	.39	.04	.01	.12	-	1.32	1.38	1.67	
D-3	17	1.34	.74	.22	1.82	.51	2.65	1.92	-	-	1.69	-	-	.021	.24	.73	-.09	-.03	.21	-	1.83	1.84	2.22	160
	20											-	-	.011	.23	.63	.09	.05	.28	-	2.13	2.16	2.21	
	21											1.9	5.0	.010	.25	.66	.13	.07	.29	.47	2.04	1.99	2.25	
	23											1.7	4.1	.015	.23	.78	-.01	.00	.23	-	1.71	1.91	2.30	
D-4	17	1.48	.79	.23	1.88	.54	2.74	1.92	-	-	2.30	-	-	.009	.24	.81	-.01	.00	.23	-	1.82	2.08	2.31	180
	20											1.7	4.5	.001	.23	.79	.22	.11	.34	-	1.88	2.24	2.41	
	21											-	-	.002	.23	.76	.16	.08	.31	.39	1.96	2.32	2.51	
	23											1.8	5.0	.004	.22	.80	.03	-.01	.21	-	1.85	2.09	2.45	
D-5	17	2.08	1.00	.28	2.09	.69	3.00	1.92	1.24	1.68	3.09	-	-	.007	.29	1.57	.25	.08	.37	-	1.33	2.00	2.47	210
	20											1.8	4.0	.002	.28	.99	.29	.15	.43	-	2.12	-	-	
	21											-	-	.016	.30	1.04	.30	.16	.45	.48	2.00	-	-	
	23											1.5	4.0	-.010	.27	1.44	.32	.11	.38	-	1.45	-	-	
D-6	17	2.44	1.11	.31	2.20	.78	3.14	1.92	1.56	1.57	2.45	1.7	3.9	.015	.32	1.55	-.19	-.07	.26	-	1.58	1.91	2.38	220
	20											2.0	4.0	.000	.31	1.10	.32	-.17	.47	-	2.22	1.98	2.28	
	21											2.0	4.0	.001	.31	1.28	.50	.26	.57	.64	1.90	2.05	2.32	
	23											1.7	4.0	-.007	.30	2.37	.90	.30	.60	-	1.03	1.95	2.30	

TABLE B-11 BASIC SCOURED DATA TEST SERIES D CONSTRICTED BED WIDTH 23"

(1) TEST NO.	(2) STATION	(3) Q_w (cfs)	(4) A_o (ft^2)	(5) d_o (ft)	(6) V_{mo} (ft/sec)	(7) A_c (ft^2)	(8) V_{mc} (ft/sec)	(9) w_{bc} (ft)	(10) A'_o (ft^2)	(11) V'_{mo} (ft/sec)	(12) C (ft^3/hr)	(13) D'_{50} (mm)	(14) D'_{90} (mm)	(15) $\triangle d$ (ft)	(16) d'_o (ft)	(17) A (ft^2)	(18) A_s (ft^2)	(19) d_s (ft)	(20) d_{av} (ft)	(21) d_{max} (ft)	(22) V_m (ft/sec)	(23) U_m (ft/sec)	(24) U_{max} (ft/sec)	(25) LENGTH OF RUN (mins)
E-1	17	·51	·39	·12	1·31	·35	1·46	2·67	-	-	·15	2·0	4·0	·002	·12	·40	·00	·00	·12	-	1·28	1·36	1·67	300
	20													·010	·13	·38	·00	·00	·13	·16	1·35	1·46	1·54	
	21													·012	·13	·38	·00	·00	·13	-	1·34	1·49	1·56	
	23													-·002	·12	·39	·01	·00	·12	-	1·31	1·42	1·55	
E-2	17	·75	·50	·15	1·49	·45	1·66	2·67	-	-	·48	2·3	4·5	·032	·18	·52	-·09	-·03	·15	-	1·43	1·64	1·78	90
	20													·033	·19	·53	-·02	-·01	·18	·22	1·42	1·73	1·81	
	21													·038	·19	·54	-·04	-·02	·18	-	1·40	1·71	1·83	
	23													·029	·16	·53	-·08	-·03	·14	-	1·41	1·69	1·83	
E-3	17	1·15	·67	·20	1·73	·60	1·92	2·67	-	-	1·60	1·8	4·2	·020	·22	·63	-·11	-·04	·18	-	1·83	1·91	2·02	135
	20													·024	·22	·65	-·03	-·01	·21	·32	1·77	1·91	2·05	
	21													·019	·22	·62	-·05	-·02	·20	-	1·87	1·94	2·19	
	23													·016	·21	·74	-·05	-·02	·20	-	1·55	1·89	2·18	
E-4	17	1·53	·80	·23	1·90	·73	2·11	2·67	-	-	2·0	1·7	4·6	-·001	·23	·75	-·05	-·02	·21	-	2·04	2·00	2·26	90
	20													·008	·24	·72	-·03	-·01	·23	·32	2·12	2·16	2·42	
	21													·006	·24	·69	-·06	-·02	·22	-	2·22	2·17	2·44	
	23													·003	·24	·79	-·02	-·01	·23	-	1·93	2·11	2·60	
E-5	17	2·48	1·12	·31	2·21	1·02	2·44	2·67	1·60	1·55	1·73	1·7	4·0	·010	·32	1·48	-·11	-·04	·28	-	1·95	1·94	2·38	120
	20													·022	·33	1·14	·04	-·02	·35	·47	1·99	2·04	2·35	
	21													·050	·36	1·14	-·07	-·03	·33	-	2·19	2·09	2·44	
	23													·032	·34	1·62	-·12	-·04	·38	-	1·53	2·06	2·52	

TABLE B-12 BASIC SCOURED DATA TEST SERIES E CONSTRICTED BED WIDTH 32"

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Test No.	Qw (c.f.s.)	Q Effective = .95 Qw for over-bank (c.f.s.)	Upstream, Mean Bed Width (ft.)	q (c.f.s./ft.)	C Observed (ppht.)	D_{50} (ft.)	do (ft.)	$\frac{do}{D_{50}}$	$\sqrt{\frac{\rho_f q^2}{\gamma_b' D_{50}^3}}$	C Predicted Figure 6 (ppht.)	W_{bc} (ft.)	q' (c.f.s./ft.)	d_{av} (ft.)	$\frac{d_{av}}{D_{50}}$ (Observed)	$\sqrt{\frac{\rho_f q'^2}{\gamma_b' D_{50}^3}}$	$\frac{h}{D_{50}}$ (Figure 6)	Predicted Scoured Depth (ft.)	Regime Depth Scoured (ft.)
B-1	.27	.27	3.0	.09	0	.0033	.08	24.8	65	1	1.42	.19	.11	33.9	137	46	.15	.17
B-2	.54	.54	3.0	.18	24.2	.0033	.12	37.6	130	30	1.42	.38	.24	73.3	274	70	.23	.26
B-3	1.13	1.13	3.0	.38	41.0	.0033	.19	58.8	274	100	1.42	.76	.33	99.4	548	100	.33	.36
B-4	1.83	1.75	3.0	.58	22.0	.0033	.26	78.5	418	100	1.42	1.29	.62	187	933	155	.51	.58
C-1	.26	.26	3.0	.09	0	.0033	.08	24.2	65	1	1.67	.15	.11	34.5	108	38	.13	.14
C-2	.67	.67	3.0	.22	24.2	.0033	.14	42.7	159	67	1.67	.40	.22	68.0	288	68	.22	.27
C-3	1.00	1.00	3.0	.33	49.0	.0033	.18	54.5	238	100	1.67	.60	.30	89.4	432	86	.28	.30
C-4	1.51	1.51	3.0	.50	46.4	.0033	.23	69.7	361	110	1.67	.90	.36	108	600	103	.34	.38
C-5	2.32	2.20	3.0	.73	27.0	.0033	.30	90.3	527	100	1.67	1.30	.55	167	933	150	.50	.58
C-6	2.86	2.70	3.0	.90	24.3	.0033	.34	102.0	650	60	1.67	1.60	.69	208	1153	190	.63	.70
D-1	.21	.21	3.0	.07	0	.0033	.07	21.5	50	1	1.92	.11	.16	47.0	79	30	.10	.12
D-2	.51	.51	3.0	.17	14.7	.0033	.12	36.4	123	30	1.92	.26	.21	62.1	187	51	.17	.18
D-3	1.34	1.34	3.0	.45	35.0	.0033	.22	65.2	325	110	1.92	.70	.29	88.0	504	90	.30	.37
D-4	1.48	1.48	3.0	.49	43.2	.0033	.23	69.1	354	105	1.92	.77	.34	104	555	100	.33	.37
D-5	2.08	1.97	3.0	.66	41.3	.0033	.28	84.0	475	120	1.92	1.00	.45	137	721	110	.36	.45
D-6	2.44	2.32	3.0	.77	27.8	.0033	.31	93.0	556	100	1.92	1.21	.60	182	873	140	.46	.56
E-1	.51	.51	3.0	.17	8.2	.0033	.12	36.4	123	35	2.67	.19	.13	40	137	40	.13	.14
E-2	.75	.75	3.0	.25	18.0	.0033	.15	46.1	181	50	2.67	.28	.18	54	202	52	.17	.18
E-3	1.15	1.15	3.0	.38	39.0	.0033	.20	59.4	274	65	2.67	.43	.21	63	310	70	.23	.26
E-4	1.53	1.53	3.0	.51	36.3	.0033	.23	70.3	364	100	2.67	.57	.23	69	411	80	.26	.32
E-5	2.48	2.35	3.0	.78	19.4	.0033	.31	94.0	559	100	2.67	.88	.38	115	633	110	.36	.39

TABLE B-13 PARAMETERS USED IN ANALYSIS AND RESULTS OBTAINED

1	2	3	4	5	6	7	8	9	10	11	12	13
Test No.	Qw (c.f.s.)	$\frac{Wbc}{Wb}$ %	UNCONSTRICTED		CONSTRICTED		$^{(7)}/_{(5)}$ Velocity Ratio	Series Average of (8)	$^{(6)}/_{(4)}$ Area Ratio	Series Average of (10)	(8) x (10)	Series Average of (12)
			A (sq.ft.)	Um (ft./sec.)	A (sq.ft.)	Um (ft./sec.)						
B-1	.27	47%	.19	1.09	.16	1.49	1.37	1.27	.84	.85	1.15	1.08
B-2	.54	47%	.37	1.48	.32	1.76	1.19		.87		1.04	
B-3	1.13	47%	.59	1.91	.53	2.44	1.28		.90		1.15	
B-4	1.83	47%	.91	1.85	.71	2.32	1.25		.78		0.97	
C-1	.26	56%	.24	1.23	.17	1.40	1.14	1.21	.71	.83	0.81	1.00
C-2	.67	56%	.43	1.57	.40	1.75	1.11		.93		1.03	
C-3	1.00	56%	.52	1.94	.47	2.15	1.11		.91		1.01	
C-4	1.51	56%	.85	1.97	.71	2.55	1.29		.84		1.08	
C-5	2.32	56%	1.72	1.90	1.09	2.49	1.31		.64		0.84	
C-6	2.86	56%	1.25	1.67	1.20	2.17	1.30		.96		1.25	
D-1	.21	64%	.28	1.29	.29	1.25	.97	1.05	1.03	.95	1.00	1.00
D-2	.51	64%	.40	1.53	.41	1.54	1.01		1.02		1.03	
D-3	1.34	64%	.73	1.84	.66	1.99	1.08		.91		0.98	
D-4	1.48	64%	.81	2.08	.79	2.32	1.12		.98		1.10	
D-5	2.08	64%	1.57	-	1.44	-	-		.92		-	
D-6	2.44	64%	1.55	1.91	1.28	2.05	1.07		.83		0.89	
E-1	.51	89%	.40	1.36	.38	1.49	1.10	1.06	.95	.94	1.05	1.02
E-2	.75	89%	.52	1.64	.53	1.73	1.06		1.02		1.08	
E-3	1.15	89%	.63	1.91	.62	1.94	1.01		.99		1.00	
E-4	1.53	89%	.75	2.00	.72	2.17	1.08		.96		1.03	
E-5	2.48	89%	1.40	1.94	1.24	2.09	1.07		.89		0.95	

TABLE B-14 DATA USED IN DEVELOPMENT OF MEAN VELOCITY METHOD

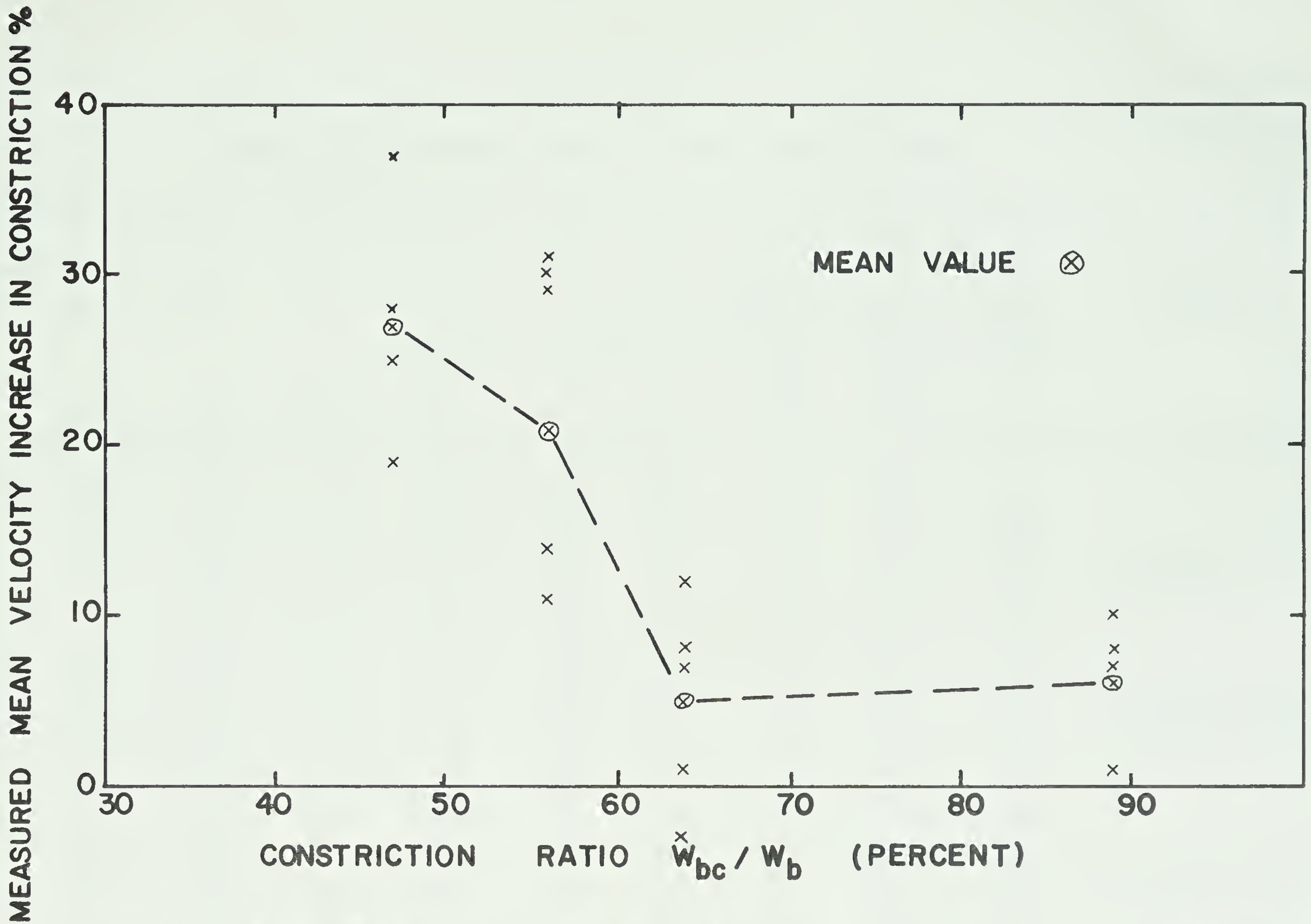

FIGURE B-3 MEASURED MEAN VELOCITY INCREASE IN CONSTRICTION VS CONSTRICTION . RATIO (FROM TABLE B-14)

TABLE B-15

CALCULATION OF SCOURED DEPTHS BY MEAN VELOCITY METHOD

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Test No.	d_o (ft)	W_{bc} (ft)	Ac (ft^2)	A Unconstricted (ft^2)	A Constricted (ft^2)	(6)-(4) Scoured Area (ft^2)	Mean Depth Below Bed (ft)	(8)+(2) (Predicted) Depth (ft)	d_{av} (Measured) (ft)	Comments
B-1	.08	1.42	.13	.19	.15	.02	.01	.09	.11	
B-2	.12	1.42	.21	.37	.29	.08	.06	.18	.21	
B-3	.19	1.42	.35	.59	.46	.11	.08	.27	.38	
B-4	.26	1.42	.50	.91	.72	.22	.15	.41	.33	Overbank Flow
C-1	.08	1.67	.15	.24	.20	.05	.03	.11	.11	
C-2	.14	1.67	.28	.43	.36	.08	.05	.19	.22	
C-3	.18	1.67	.37	.52	.42	.05	.03	.21	.24	
C-4	.23	1.67	.49	.85	.70	.21	.13	.36	.36	
C-5	.30	1.67	.67	1.77	1.42	.75	.45	.75	.55	Overbank Flow
C-6	.34	1.67	.79	1.25	1.00	.21	.13	.47	.60	Overbank Flow
D-1	.07	1.92	.15	.28	.26	.11	.06	.13	.15	
D-2	.12	1.92	.26	.40	.38	.12	.06	.18	.20	
D-3	.22	1.92	.51	.73	.69	.18	.09	.31	.29	
D-4	.23	1.92	.54	.81	.76	.22	.11	.34	.31	
D-5	.28	1.92	.69	1.57	1.48	.79	.41	.69	.45	Overbank Flow
D-6	.31	1.92	.78	1.55	1.46	.68	.35	.66	.57	
E-1	.12	2.67	.35	.40	.38	.03	.01	.13	.13	
E-2	.15	2.67	.45	.52	.49	.04	.02	.17	.18	
E-3	.20	2.67	.60	.63	.59			d_o=.20	.21	
E-4	.23	2.67	.73	.75	.70			d_o=.23	.23	
E-5	.31	2.67	1.02	1.48	1.40	.38	.14	.45	.35	

Appendix C

PLATES

PLATE 1
GENERAL VIEW OF MODEL LOOKING UPSTREAM

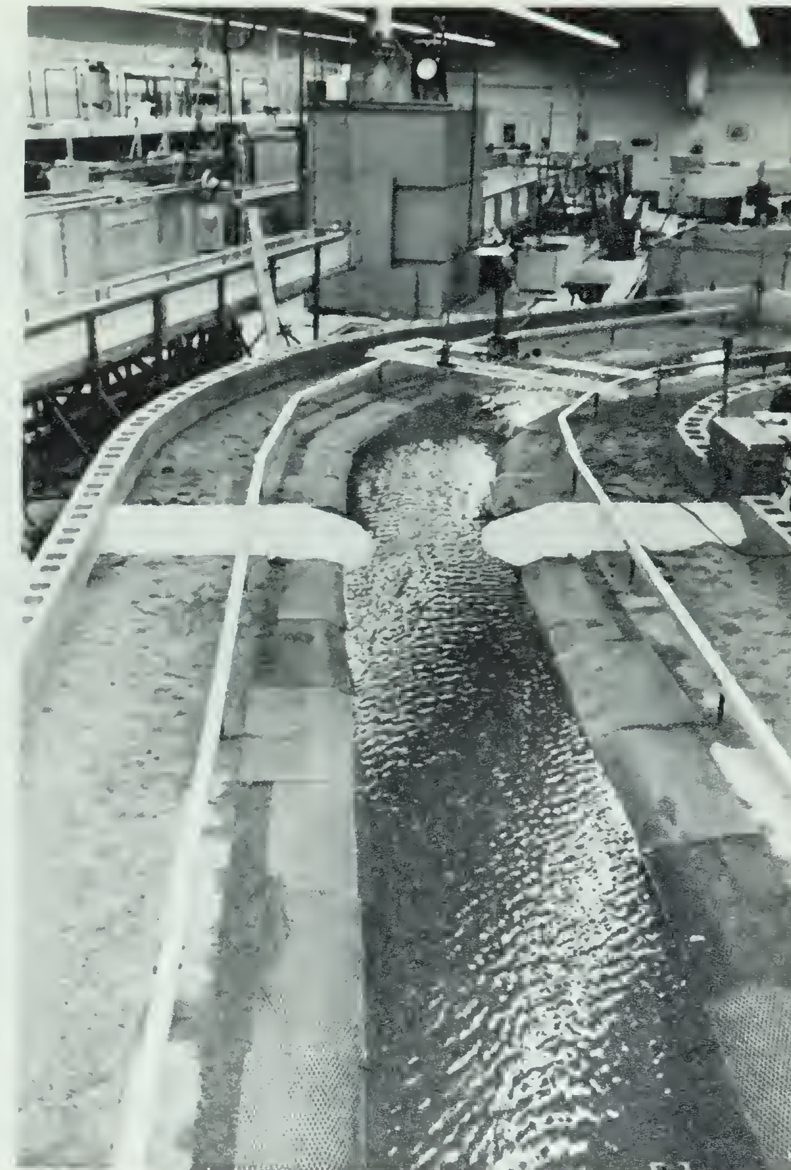

PLATE 2
GENERAL VIEW OF MODEL LOOKING DOWNSTREAM

PLATE 3
VIEW OF SCOUR HOLE Q_w 1.51 C.F.S.

PLATE 4
VIEW OF SCOUR HOLE Q_w 2.32 C.F.

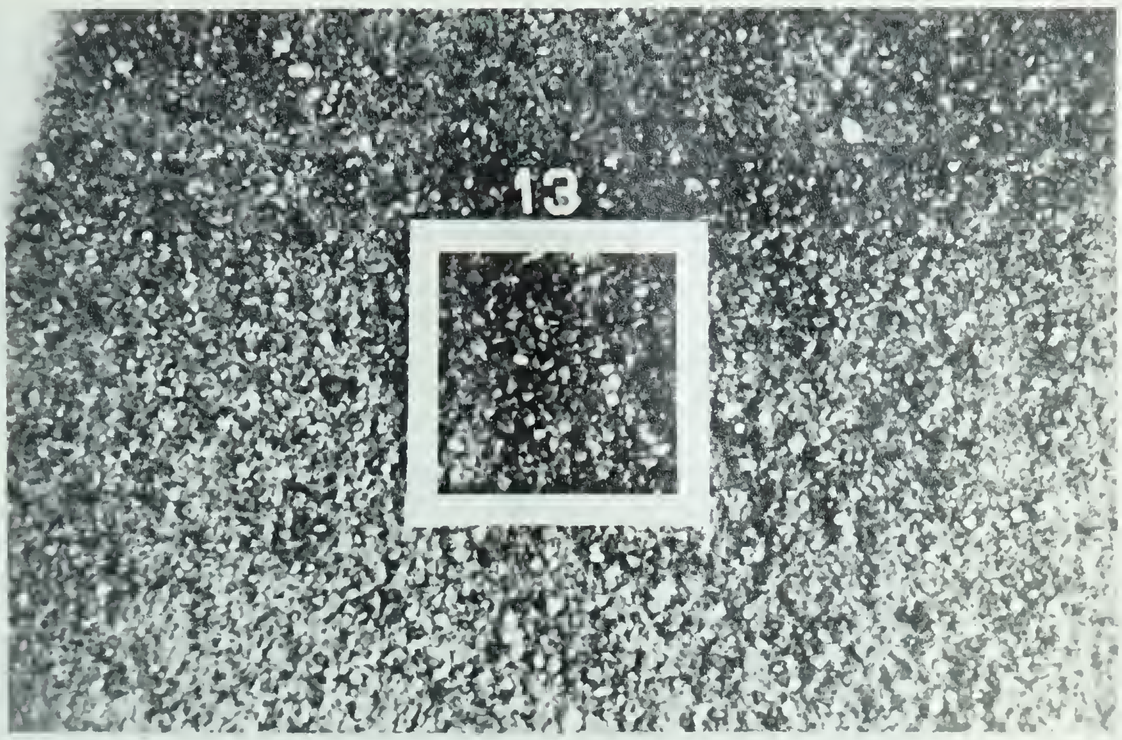

PLATE 5
BED PAVING AT STATION 13, Q_w 0.13 C.F.S.
[INSIDE OF SQUARE GRID IS 4 INCHES]

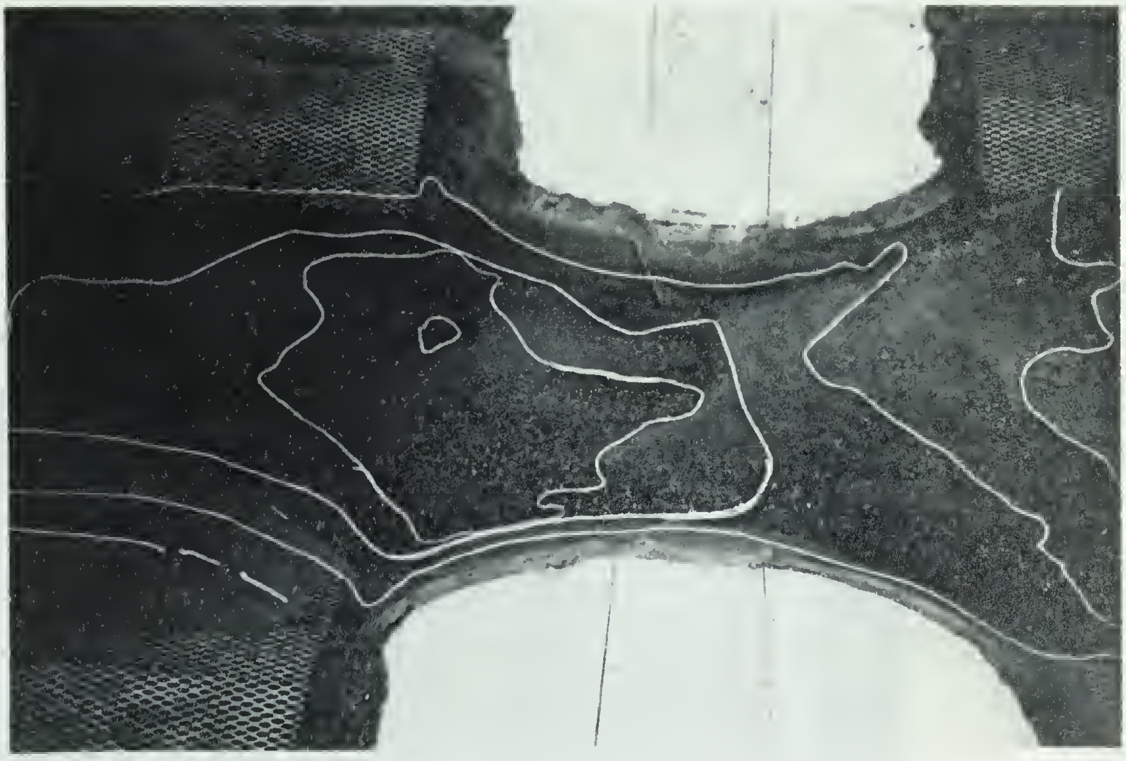

PLATE 6
PLAN VIEW OF TYPICAL SCOURED HOLE
CONTOUR INTERVAL 0.1 FEET
[FLOW DIRECTION FROM RIGHT TO LEFT]

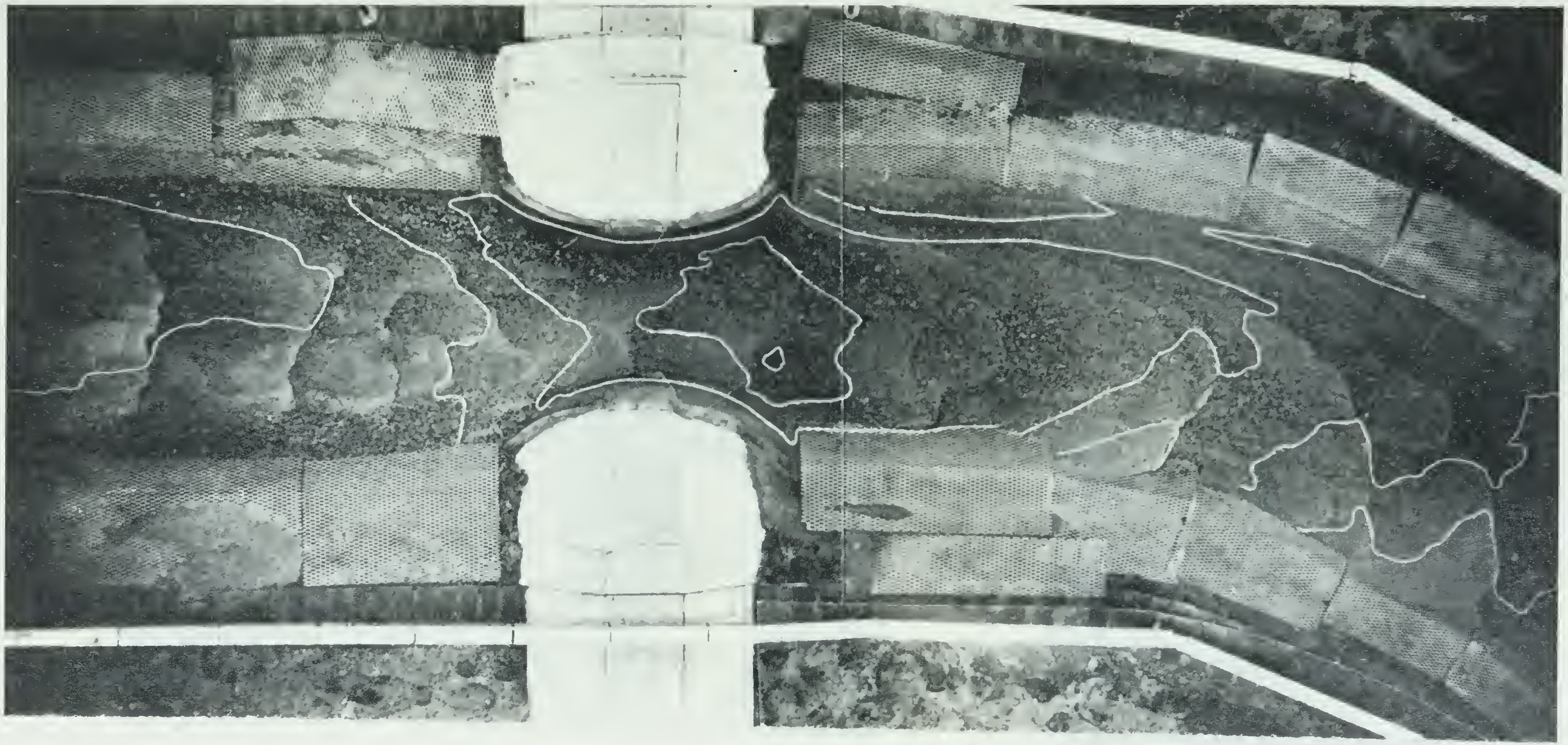

PLATE 7

COMPOSITE VIEW OF TYPICAL SCOURED HOLE

[CONTOUR INTERVAL 0.1 FT.]

PLATE 8
OVERBANK FLOW THROUGH MODEL
Q_w 2.32 C.F.S.

PLATE 9
NORMAL FLOW CONDITIONS
Q_w 0.25 C.F.S.